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THESIS ON CRITICAL EVALUATION OF ETHIOPIAN BUILDING CODE STANDARD 2

BY

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Abbreviation:

Note: The abbreviation used is based on ISO 3898:1987

Latin upper case letters

A	Accidental action
A	Cross sectional area
Ac	Cross sectional area of concrete
As	Cross sectional area of reinforcement
As,min	minimum cross sectional area of reinforcement
Asw	Cross sectional area of shear reinforcement
D	Diameter of mandrel
DEd	Fatigue damage factor
E	Effect of action
Ec,Ec(28)	Tangent modulus of elasticity of normal weight concrete at a stress of σc = 0 and at 28 days
Ec,eff	Effective modulus of elasticity of concrete
Ecd	Design value of modulus of elasticity of concrete
Ecm	Secant modulus of elasticity of concrete
Ec (t)	Tangent modulus of elasticity of normal weight concrete at a stress of σc= 0 and at time t
Es	Design value of modulus of elasticity of reinforcing steel
EI	Bending stiffness EQU Static equilibrium
EQU	Static equilibrium
F	Action
Fd	Design value of an action
Fk	Characteristic value of an action
Gk	Characteristic permanent action
I	Second moment of area of concrete section
L	Length
M	Bending moment
MEd	Design value of the applied internal bending moment
N	Axial force
NEd	Design value of the applied axial force (tension or compression)
Qk	Characteristic variable action
Qfat	Characteristic fatigue load
R	Resistance
S	Internal forces and moments

S	First moment of area
SLS	Serviceability limit state
T	Torsional moment
TEd	Design value of the applied torsional moment
ULS	Ultimate limit state
V	Shear force
VEd	Design value of the applied shear force

Latin lower case letters

a	Distance
a	Geometrical data
Δa	Deviation for geometrical data
b	Overall width of a cross-section, or actual flange width in a T or L beam
bw	Width of the web on T, I or L beams
d	Diameter; Depth
d	Effective depth of a cross-section
dg	largest nominal maximum aggregate size
e	Eccentricity
fc	Compressive strength of concrete
fcd	Design value of concrete compressive strength
fck	Characteristic compressive cylinder strength of concrete at 28 days
fcm	Mean value of concrete cylinder compressive strength
fctk	Characteristic axial tensile strength of concrete
fctm	Mean value of axial tensile strength of concrete
f0.2k	Characteristic 0.2% proof-stress of reinforcement
ft	Tensile strength of reinforcement
ftk	Characteristic tensile strength of reinforcement
fy	Yield strength of reinforcement
fyd	Design yield strength of reinforcement
fyk	Characteristic yield strength of reinforcement
fywd	Design yield of shear reinforcement
h	Height
h	Overall depth of a cross-section
i	Radius of gyration
k	Coefficient; Factor
l	(or l or L) Length; Span
m	Mass
r	Radius

1/r	Curvature at a particular section
t	Thickness
t	Time being considered
t0	the age of concrete at the time of loading
u	Perimeter of concrete cross-section, having area Ac
u,v,w	Components of the displacement of a point
x	Neutral axis depth
x,y,z	Coordinates
z	Lever arm of internal forces

Greek lower case letters

α	Angle; ratio
β	Angle; ratio; coefficient
γ	Partial factor
γA	Partial factor for accidental actions A
γC	Partial factor for concrete
γF	Partial factor for actions, F
γF,fat	Partial factor for fatigue actions
γC,fat	Partial factor for fatigue of concrete
γG	Partial factor for permanent actions, G
γM	Partial factor for a material property, taking account of uncertainties in the material property itself, in geometric deviation and in the design model used
γQ	Partial factor for variable actions, Q
γf	Partial factor for actions without taking account of model uncertainties
γg	Partial factor for permanent actions without taking account of model uncertainties
γm	Partial factors for a material property, taking account only of uncertainties in the material property
δ	Increment/redistribution ratio
ζ	Reduction factor/distribution coefficient
εc	Compressive strain in the concrete
εc*1*	Compressive strain in the concrete at the peak stress fc
εcu	Ultimate compressive strain in the concrete
θ	Angle
λ	Slenderness ratio

μ Coefficient of friction between the tendons and their ducts

ν Poisson's ratio

ν Strength reduction factor for concrete cracked in shear

ρ Oven-dry density of concrete in kg/m3

ρl Reinforcement ratio for longitudinal reinforcement

ρw Reinforcement ratio for shear reinforcement

σc Compressive stress in the concrete

σcu Compressive stress in the concrete at the ultimate compressive strain εcu

φ Diameter of a reinforcing bar

ϕn Equivalent diameter of a bundle of reinforcing bars

$\varphi(t,t0)$ Creep coefficient, defining creep between times t and t0, related to elastic deformation at 28 days

$\varphi(\infty,t0)$ Final value of creep coefficient

ψ Factors defining representative values of variable actions

$\psi 0$ for combination values

$\psi 1$ for frequent values

$\psi 2$ for quasi-permanent values

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ABSTRACT

Design of a concrete structure is a step by procedure beginning with estimation of the loads acting on the structural system, preliminary proportioning of relative stiffness's of the structural system, carrying out a detailed and often iterative general structural analysis, followed by the sizing or proportioning of the members and concluding with the detailing process. While engineers are usually well trained in analysis procedures and the basic mechanics of structural concrete, there is not a general methodology for detailing. This often presents the designer with numerous difficulties. The different codes and standards propose empirical recommendations for some specific applications. However the design standards cannot include the innumerable details that may arise.

Structural design and construction has the objective to build safe durable, serviceable, economical and aesthetic structures. The translation of design into correspondingly high quality structure necessitates good detailing and construction practices. The limit state method design primary aim is to minimize the probability of failure to an acceptable low value. In this context attainment of limit state is called failure. Limit states imply those conditions where by a structure ceases to fulfill the functions for which it has been designed. By using limit states method (allowing for inelasticity and moment redistribution) with partial safety factors, with higher strength materials, for higher stresses, modern structures are designed. And also methods of analysis and design have become more sophisticated and accurate and the benefit of conservation in built in approximate methods is no longer available. Therefore these results in taller structures and longer spans more slender members and thinner slabs and walls and are built at faster pace.

They are therefore more flexible (in terms of deflections) and are more crack prone as compared with the old structures which used to be low in height had thicker(stockier) members were lightly stressed and were built at slower place. Thus serviceability criteria assume for greater importance in modern structures. As a result of the introduction of limit state method of design for reinforced

concrete structures and the concept of development length has become extremely important as many of the design requirements are to be met through detailing.

The design and building processes would subjected to substantial impact from construction details. Also it affects concrete structure durability and maintenance. Around concrete structures the problems arising are attributable to design stage and due to lack of or errors in construction details. Their share is 50% and 50%.

Knowledge in four areas synthesized for good detailing:

1) Knowledge on structural concrete engineering underlying theory
2) Professional practice in on-site.
3) Laboratory trials information from experiments.
4) Forensic engineering studies experience.

Detailing practices, construction practices, quality control in construction, building failures causes and prevention of cracks/leakage in buildings etc. are all large enough topics. Not all these topics strictly within the scope of this book (Detailing manual). As such an attempt is made here only to draw attention to some of the major and most common causes of failure pertaining to the design(and construction) of reinforced concrete buildings. In Detailing generally included are the properties of single and bundled bars, selection of beam reinforcement, and crack control requirements, development length of bars and minimum radii of bends. Therefore this book will be useful to concrete design engineers, field engineers and students of civil engineers.

Key words: Relative Stiffness, Detailing, mechanics of structural concrete, Durable, codes, limit state, Forensic engineering

Chapter-1

Introduction

1.1. Introduction

In the beginning of year 1995(G.C) Ministry of works and urban development prepared Building code standards of general application. The purpose of these standards to serve as nationally recognized documents, the application of which is deemed to ensure compliance of buildings with the minimum requirements for design, construction and quality of materials set down by the "National Building Code".

The major benefits to be gained in applying these standards are harmonization of professional practice and ensuring of appropriate levels of safety, health and economy with due consideration of the objective conditions and needs of the country.

Intended response of the structure from the safety, health and economy can be achieved essentially by providing proper detailing .And also important that fundamental theory and required structural behavior of reinforced concrete structures are known in order to reduce the risk of incorrect interpretations and unacceptable solutions. Therefore it is of great interest to know the background of EBCS-2 in order to implement the rules in correct manner. And also it is important to investigate how the structural engineers are interpreting rules and applying them to the practical conditions.

1.2.General behavior of reinforced concrete structures:

In order to know how to design a structural member and what analysis approaches that are valid, it is important to understand the typical response of the structural member in reinforced concrete.This response can be divided into four different stages.

1) Un cracked stage
2) Cracked stage
3) Yielding and

4) Collapse

1)Un cracked stage

In the un cracked stage the influence of reinforcement is small(i.e., homogeneous material can normally be assumed in analysis). The relation between the load and deformation is linear elastic (i.e., in case of bending the curvature increases linearly with the applied bending moment).

The linear elastic analysis results one unique solution independent of load, hence it is only the magnitude of stresses that increases with increased loading. Since the sections are un cracked the stresses is constant even when the load increases but the shape of the stress field remains (i.e., Global response also is be linear elastic)

2) Cracked

Since stiffness depends on whether the section is cracked or un cracked, there will be drastic changes of stiffness when cracking occur. In the cracked regions the stiffness is dependent on the reinforcement amount and its arrangement. In this stage the relation between the load and deformation is no longer linear.

In statically indeterminate structures stiffer regions attract forces hence redistribution of moment takes place, stress redistribution due to cracking. The movement thus decreases in the cracked regions and increases in the un cracked regions and influences also the global response.

As the load increases cracking of the structural member continue until the member becomes fully cracked. As a result the stress field configuration differs from that in the un cracked state.

Due to cracking and continuous change of stiffness distribution, reinforced concrete members have a nonlinear response in the cracked stage. The ultimate stage is reached when one of the materials concrete or steel reaches nonlinear behavior, for instance when the reinforcement starts to yield in a region of the member.

3) Yielding and collapse

Yielding of a region will drastically affect the response of that region as well as the global response. However this does not mean that the capacity of the member reached. For statically indeterminate structures the load can still be increased due to plastic redistribution even though significant increase of the moment in the yielding section is not possible .The plastic redistribution due to the plastic deformation of the yielding region which behaves like a plastic hinge. When load increases one or more plastic regions will develop and when plastic resistance is reached in some critical regions it determines the resistance of the whole structural member. A collapse mechanism forms when a critical member of plastic hinges develop. However a condition is that the needed plastic rotations in the first hinge(s) can develop. Otherwise a premature failure a hinge occurs before a collapse mechanism is formed. To analyze the equilibrium conditions in the collapse stage an ideally plastic behavior of the sections can be assumed and the force distributions can be solved by means of theory of plasticity.

1.3 .Statement of the problem:

In order to fill the gaps of EBCS-2 with EBCS draft codes by comparing with other codes and to facilitate the use of EBCS-2. Along with diagrammatic representation.

1.4.Objectives:

1.4.1. General objective

- To fill the ambiguities and gapsof EBCS code..

1.4.2. Specific objectives.

- To develop a detailing guide
- To understand the background for the rules of EBCS-2
- To investigate the current practice of EBCS-2 in applying to the structures.

1.5.Methodology

A more detailed literature study to look into the background of the highlighted areas looked into .Literature search among articles and publications. To get further information and increase the understanding codes and related publications from other countries codes like EURO, ACI, AS and IS.

1.6.Scope:

In depth understanding of the EBCS codes provisions , identify the ambiguities and gaps in the EBCS code , those ambiguities and gaps will be filled with literature review and studying the developed countries procedures researched and reported. And this study is aimed to develop a relatively simple approach for detailing.

This detailing code book is concentrated to the common structural elements like Beams, slabs, columns and walls, along with foundations. And also drawn the attention of the ultimate limit state and serviceability limit state. In order to enhance the quality of this thesis while explaining the background for various clauses , the concepts are correlated with the advanced codes like European code(EC), American concrete institute(ACI) code, Australian code(AS) and Indian standard(IS) code for those not provided by the EBCS code.

1.7.Limitations:

- No practical experiments or numerical modeling using FE programs have been executed
- Plain, Pre stressed and prefabricated concrete should be left out in order to reduce the numberof issues into a manageable amount.

1.8.Literature review:

Robin whittle (2013), Failures in concrete structures (case studies in reinforced and pre stressed structures) revealed various causes of failuresof the reinforced concrete structures, they are particularly related to

- Failures due to misuse of code of practice clauses
- Failures due to inappropriate extrapolation of code of practice clauses
- Problems and failures due to poor detailing etc.

In this he found that poor detailing is often connected with a lack of sufficient design thought and accompanied by poor workmanship in construction .The combination can lead to structural failure.

He made a remarkable comment on codes suggested minimum reinforcement and it's relation with cracking. Codes of practice stipulate a minimum amount of reinforcement based on the tension strength of the concrete. But the tension yield strength of the reinforcement should be at least equal to the tension strength of the concrete when minimum reinforcement is provided and it related to cracking also.

My study aimed to fill EBCS gaps and develop a relatively simple approach for detailing.

Chpater-2

Basic Design Principles

2.1. Introduction

The aim of limit state design is to ensure that the structure being designed remains safe and serviceable throughout its life and what it does not become unfit for the use for which it was intended, i.e., it does not reach a “limit state” by violating one or more of the criteria governing it’s performance and use.

The limit state associated with the maximum load carrying capacity of the structure i.e., its safety is called the limit state of collapse or the ultimate limit state whereas those associated with the performance of the structure under service loads are called the limit states of serviceability.

The principal limit states to be checked for in design correspond to the following conditions.

1) Limit state of collapse

   The whole structure or part of it may collapse as a result of failure one or more critical sections, elastic or plastic instability, overturning, fatigue effects etc. Design for the ultimate strength in flexure, shear etc. corresponds to this limit state.

2) Limit state of deflection:

   The structure may deflect excessively, adversely affecting its serviceability performance

3) Limit state of cracking

   Excessive cracking of concrete may be determined to the appearance and integrity of the structure and may lead to corrosion of reinforcement.

In addition to these limit states, in special cases the structure may have to be checked for other effects such as vibration, durability, fire resistance etc.

Any assessment of the safety and serviceability of a structure must take into account the variations in the imposed loads and in the strengths of the materials of which the structure is composed as well as the inadequacies in the methods of analysis, design and construction. Safety and serviceability can therefore be meaningfully expressed only in terms of the probability that the structure will not reach a limit state. Accordingly the design process must ensure that the probability of a limit state being attained is kept at an acceptably low value for the type of structure in question.

However such a strict probabilistic design procedure is not possible at present principally because on the one hand there is a lack of statical data concerning the design variables and on the other a number of effects not readily amenable to statcial treatment modify the loading and the resistance of the members even where these are derived statically. The design approach recommended in the code is therefore “Semi-Probabilistic” i.e., one which treats only the basic design variables in a probabilistic manner and covers the other uncertain influences through “partial safety factors”. This is achievedthrough the concept of “characteristic” and “design values” as discussed below.

2.2. Basic design Parameters:

The design process essentially consists in ensuring that the resistance of a member , expressed in terms of the strengths of its component materials, is greater than the forces acting on it, expressed in terms of the loads. Hence the design data may be divided into two basic groups:

1) For loads
2) The material strengths.

2.2.1. Characteristic values:

These account for the normal statical variations in the basic design data and are expressed as Characteristic value = mean value (k * Standard deviation)

Where the factors "k" ensures that the probability of the actual values being either greater(as in the case of loads) or smaller(as in the case of material strengths) than the characteristic value is kept at an acceptably low level.

For material strengths, that code defines the characteristic value as that below which not more than 5% of the test results fall. This definition implies a value of k=1.64. Accordingly the characteristic strength of materials is given by the following expression

Characteristic strength(clause 3.12(1))

Mean strength

Standard deviation

The analogues definition of characteristic load would be difficult to operate in practice in the absence of complete statically data on the variability of loads. Hence for the present it is necessary to assume that the characteristic load is that specified in the appropriate loading standard as stipulated in the code.

2.2.2. Design values:

A number condition such as accidental overloads and errors in design and construction modify adversely the load effects and member resistances so that their eventual magnitudes in the actual structure are different from those derived on the basis of the characteristic values. Such effects are accounted for by modifying the characteristic values through "Partial Safety factors" for each limit state, to give the design values appropriate to that limit state. Thus

Design strength of material =

Design Load =

Where

Partial safety factor for materials

Partial safety factor for loads.

It is these design values that should be considered in formulating the design equations.

2.2.2.1 .Concrete

Design compressive and tensile strengths (Clause 3.16)

(1) The value of the design compressive strength is defined as

Where:

is the partial safety factor for concrete, and

is the coefficient taking account of long term effects on the compressive strength and of unfavorable effects resulting from the way the load is applied.

Note: For use of αcc refer to the national annex,

The recommended value is 0.85.

EC recommends a value of1.

(2) The value of the design tensile strength,is defined as

Where:

is the partial safety factor for concrete, and

is a coefficient taking account of long term effects on the tensile strength and of unfavorable effects, resulting from the way the load is applied.

Note: - For use of refer to the national annex,

The recommended value is 0.85.

EC recommends a value of1.

For concrete Poisson's ratio may be taken =0.2 for uncracked concrete and "0" for cracked concrete. (Clause 3.13(4))
Unless more accurate information is available, the linear coefficient of thermal expansion may be taken equal to (Clause 3.13(5)

Compressive strength class: Compressive strength class defines 28 days compressive strength C32/40 means

i) Letter for type of concrete

C- Normal and heavy weight

LC- Light weight

ii) 32- Minimum characteristic of 150 mm dia by 300mm cylindrical strength

iii) 40-Minimum characteristic cube strength

Concrete Modules of Elasticity: (From Table 7.2 of BS 8110 Part-2, 1985):
Typical mean values for the static modules of elasticity at 28 days for normal weight concrete are given
Typical range for the static modules of elasticity at 28 days of normal weight of concrete

	,	
	Mean value	**Typical range**
20	24	18 to 30
25	25	19 to 31
30	26	20 to 32
40	28	22 to 34
50	30	24 to 36
60	32	26 to 38

Table-2.0Concretemodules of elasticity

2.2.2.2. Reinforcing steel:

General: (Clause- 3.2.1)

1) Reinforcement which is in the form of bars, de coiled rods, welded fabric and lattice girders. Not related to specially coated bars.

2) The required properties of reinforcing steels shall be verified using a yield strength, which relates to the characteristic values.

3) is the characteristic yield stress based on only that reinforcement used in particular structure.

4) The application rules relating to lattice girder apply only to those made with ribbed bars. Lattice girders made with other types of reinforcement may be given in an appropriate Ethiopian Technical Approval.

5) The application rules for design and detailing in this code of practice are valid for a specified yield strength range (Clause 3.2.2(3))

6) The reinforcement shall have adequate ductility as defined by the ratio of tensile strength to the yield stress, (. (Clause 3.2.4(1))

7) Design assumption: Design should be based on the nominal cross section area of the reinforcement and the design values derived from the characteristic values(Clause 3.2.7(1))

8) The mean value of density may be assumed be 7850 Kg/m^3 (Clause 3.2.7(3))

9) The design value of the modulus of elasticity may be assumed to be 200 Gpa.(Clause 3.2.7(4))

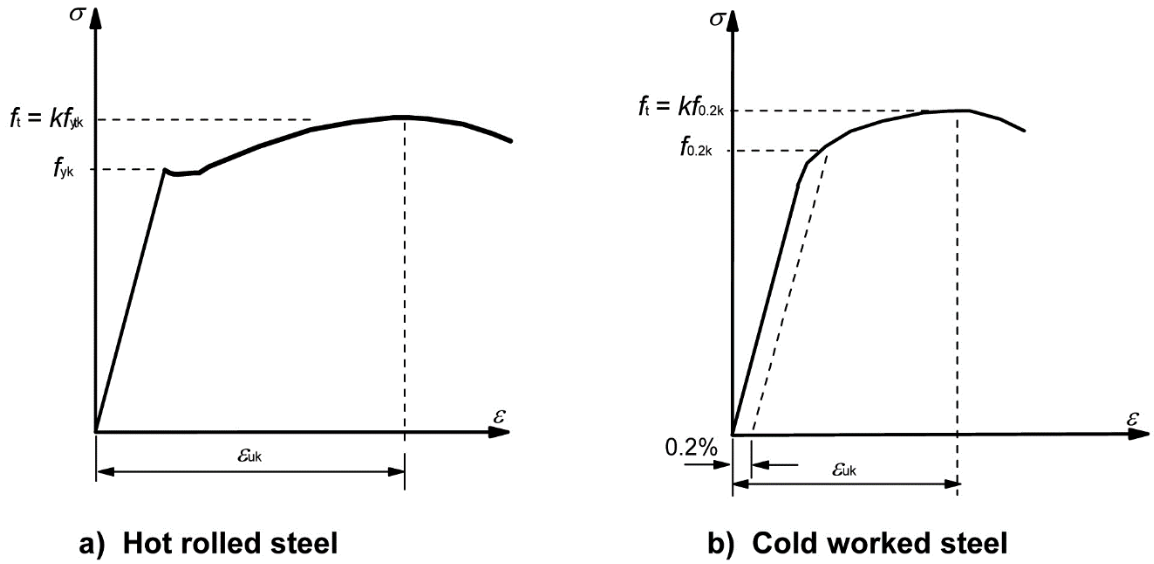


Figure2.0.Stress-strain diagrams of typical reinforcing steel (absolute values are shown for the tensile stress and strain

Economy in use of steel:

The type of steel used is generally specified by the designer but bear in mind that up to rd of the mass of steel can be saved by using high tensile steel instead of mild steel. Furthermore as the rates for small diameters are higher than those for large diameter, it is desirable to use the largest available size of bar within the design requirements. Larger bars are also produce stiffer cages and are not displaced.

2.2.2.3. Partial factors:

1) For Fatigue loads(Clause 2.4.2.3(1))

   The partial factor for fatigue loads

2) For Materials (clause 2.4.2.4(1))

a) Ultimate limit state: (Table 2.1N, EBCS-2 partial factors for materials for ultimate limit states)

Design situation			
Persistent and Transient	1.5	1.15	1.15
Accidental	1.2	1.0	1.0

Table-2.1, Partial factors for materials for ultimate limit states

There are three types of design situations:

i) Persistent- Corresponding to normal use

ii) Transient- For example during construction, refurbishment or repair

iii) Accidental- Such as fire, explosion or earthquake.

In fatigue verification the partial factors for persistent design situations given in table 2.0 are recommended for the values of

b) Serviceability limit state

not covered by EBCS is 1.0

Partial factors for material (For Foundations)

For concrete () - for calculation of design resistance of cast in place without permanent casing.Values of above table multiplied by

=1.1(National Annex.)

Extract from NAD Table-2:

Variable				

Action	Building type			
Imposed floor loads	Dwellings	0.5	0.4	0.2
	Other occupancy classes	0.7	0.6	0.3
	Parking	0.7	0.7	0.6
Imposed roof loads	All occupancy classes	0.7	0.2	0.0
Wind loads	All occupancy classes	0.7	0.2	0.0

Table-2.2 Combination factors (:

Note: NAD: National Application Document which is issued by the member countries (BSI)

2.3. Differential settlements/movements:-

Differential settlements/movements of the structure due to soil subsidence should be classified as permanent action G_{set} , which is introduced as such in combinations of actions

Note: Where differential settlements are taken into account appropriate estimate values of predicted settlements may be used.

2.3.1. Ultimate limit states:

It is considered

1) Fatigue conditions

2) In the verification of stability where second order effects are of importance etc. It is not considered in other cases provided that the ductility and rotation capacity of the elements are sufficient

2.3.2. Serviceability limit state:

The effects of differential settlements should generally be taken into account for the verification for serviceability limit states

Where differential settlements are taken into account a partial safety factor for settlement effects should be applied. (Safety factors Annex of EBCS EN 1990:2013)

2.4. Shrinkage and creep:

2.4.1 Serviceability limit state:

Shrinkage and creep are time dependent of concrete. Their effects should generally be taken into account for the verification of serviceability limit states.

2.4.1.1. Creep:

Where a stress is applied to a concrete specimen and kept constant the specimen shows an immediate strain followed by a further deformation which progressing at a diminishing rate may become several times the original immediate strain. The immediate strain is often referred to as the elastic strain is often referred to as the elastic strain and the subsequent time dependent strain referred to as the creep strain or simply the creep. That part of strain which is immediately recoverable upon removal of the stress is called the elastic recovery and the delayed recovery the creep recovery. The elastic recovery is less than the elastic strain the creep recovery is much less than the creep.

2.4.1.2. Shrinkage of concrete:

Drying of concrete in air results in shrinkage, while concrete kept under water swells. When the change in volume by shrinkage or by swelling is restrained stresses develop. In reinforced concrete structures the restraint may be caused by the reinforcing steel by the supports or by the difference in volume change of various parts of the structure. The symbol will be used for the free (Unrestrained) strain due to shrinkage or swelling. In order to comply with the sign convention for other causes of strain is considered positive when it represents elongation. Thus shrinkage of concrete is negative quantity. Stresses caused by shrinkage are generally reduced by the effect of creep of concrete. Thus the effects these two simultaneous phenomenon must be considered in stress analysis. For this purpose the amount of free shrinkage and an expression for its variation with time are needed.

Shrinkage starts to develop at a time " when moist curing stops.

The strain that develops due to free shrinkage between and a later instant "t" may be expressed as follows.

Total shrinkage that occurs after concrete hardening up to time infinity.--- it epends upon the quality of concrete the ambient air humidity.

2.4.2.Partial factor for shrinkage action:

Where consideration of shrinkage action is required for ultimate limit state a partial factor should be used. (Clause 2.4.2.1 EBCS draft code)

2.4.3. Ultimate limit states:

Shrinkage and creep to be considered

1) In the verification of ultimate limit state of stability where second order effects are of importance.

Shrinkage and creep need not be considered.

1) In other cases provided that ductility and rotation capacity of the elements are sufficient.

When creep is taken into account its design effects should be evaluated under the quasi permanent combination of actions irrespective of the design situation considered i.e. persistent , transient or accidental(clause 2.3.2.2)

In building structures temperature and shrinkage effects may be omitted in Global analysis provided joints are incorporated at every distance “d_{joint}“to accommodate resulting deformations. The recommended value of d_{joint}=30m (cast-in-situ structures)(given in National Annex)(clause 2.3.3(3))

Chapter-3
Bond and Anchorage:

3.1. Introduction

An anchorage failure is when the reinforcement is detached or released from the concrete such as splitting or pulls out failure. Reinforcement in a part of a concrete member that is subjected to tensile stresses must be sufficiently anchored in order to provide for a ductile structural behavior. Sufficient anchorage means that the reinforcing bars are placed with an extension beyond the considered section that is long enough to build up a capacity equal to the applied force in this section. The force at the end of a bar must always be zero. The force increase in a reinforcement bar is achieved by bond stresses between the reinforcement steel and the surrounding concrete along a distance called transmission length.

The length that is required in order to achieve the needed tensile capacity in a section is the required anchorage length and is thus dependent on both the load effect and maximum force increase per unit length.

The maximum force increase will be exceeded if the provided anchorage length is too short, therefore the reinforcing bars will be detached or released from the concrete.

The maximum force increase per surface unit area " of the reinforcing bar and is limited by the strength of concrete represented by the bond strength. At design of anchorage zones it is normally assumed that the bond stress at all sections along the anchorage length is equal to the bond strength

Fig-3.0(a)

Development length

Fig-3.0(b)

Fig-3.0. Relation between the required and maximum anchorage length.

Fig-3.1(a)

Fig-3.1(b)

Fig-3.1.Distribution of bond stress along the transmission length

When the reinforcement in a structure is curtailed it is important to also consider the need for anchorage. The reinforcement must be placed such that the force for which is intended to be resisted by the bar can develop.

For reinforcement bar to reach its yield stress at a critical cross section a minimum lengthof reinforcing bar(an anchorage) is required on either side of the section.

AS 3600-2009- Specifies a minimum length called the development over which a straight bar in tension must be embedded in the concrete in order to develop yield stress. An average design ultimate bond stress is assumed at the interface between the concrete and the reinforcing bar(
Depends on

i) Type and condition of reinforcing bar
ii) Strength and compaction of concrete
iii) Concrete cover and bar spacing
iv) Transverse reinforcement, Transverse pressure(Tension)

Use adequately anchored reinforcement wherever a tensile force is required for equilibrium. Sufficient bottom steel must be anchored for a length past the midpoint of the bearing to develop a tensile force of

$V\cot\theta_v/\emptyset$ (Plus any additional force arising from restraint)

Development length is the embedded length of the reinforcement required to develop the design strength of the reinforcement at critical section. Critical section for development of reinforcement in flexural members is at points of maximum stress and at points within the span where adjacent reinforcement terminates or is bent.

3.2. Anchorage of bottom reinforcement at end supports:

Requirements in Euro Code-2:

Section EC 29.2.1.4. of Eurocode-2 addressed specifically anchorage of bottom reinforcement at end supports. According to this clause at least 25% of the amount of reinforcement requirement in the span section of a beam should be provided as bottom reinforcement at supports with little or no end fixity.

For slabs the curtailment and anchorage may be carried out as for beams. However for simply supported slabs the amount of reinforcement required in the bottom of support sections should be at least 50% of the reinforcement in the span according to EC 29.3.1.2(1)

The design of anchorage length " at supports is according to section EC 29.2.1.4 (3) measured from the intersection point between beam and support.

The reinforcement is cutoff that is opposite of curtailment is splicing (when reinforcing bars have to be spliced longitudinally in order to cover the whole length of the structure) lapping is one of the most frequently used among different ways to enable splicing of bars. As the transmission of forces between reinforcement and surrounding concrete in case of anchorage the transmission of forces between bars in lap joint is also similar to the bars in a lap splice do not have to be in close contact to each other. The load path is enabled through the concrete between the bars in a truss like system.

Fig-3.2(a)

Maximum force increase per unit length

Fig-3.2(b)

Fig-3.2.Transfer of forces between bars in a lap splice

The same type of failures i.e., anchorage failures must be considered in case of a lap splice .The contact forces for bond between steel and concrete, inclined compressive forces with transverse and longitudinal components. The lap length must be long enough to build up tensile capacity equal to the tensile force that needs to be transferred through the lap. This means that lap length is dependent of the basic required anchorage length

Explanation for the fig-3.2: Because of the inclined stress field between the bars there will be transverse component that tries to push the reinforcing bars away from each other creating tensile forces which may result in splitting cracks in concrete surrounding the splice zone. A so called splice failure can also occur due to spalling of concrete. However the transverse forces can be balanced by transverse reinforcing barsthat thus are of great importance for the capacity of the lap splice.

3.3. Lapping of longitudinal reinforcement: According to E.C-2 section 28.7.1 splicing can be established by lapping, welding or mechanical devices, lap splices require careful reinforcement detailing. In some reinforcement configurations such as torsional links or stirrups splicing might also be necessary.

Splicing is able to create reinforced concrete members longer than available reinforcement bars, it is necessary to splice reinforcement such that forces can be transmitted between the bars.

In order to prevent splitting failures of concrete and avoid large cracks, to ensure transmission of forces between bars, lap splices must be arranged with certain distances to each other both longitudinally and transversally. The bars that are spliced together should also be placed within a certain distance to each other.

3.3.1. Laps and mechanical couplers

General

(1)Lapping or splicing is required to transfer force from one bar to another. The forces are transferred from one bar to the other through bonds in concrete. Force is first transferred to the concrete through bond from one bar and then it is transferred to the other bar forming the splice through bond between it and concrete. Thus concrete at the point of splicing is subjected to high shear and splitting stresses which may cause cracks in concrete. Methods of splicing include

i) Lapping of bars, with or without bends or hooks;
ii) Welding;
iii) Mechanical devices assuring load transfer in tension-compression or in compression only.(Mechanical couplers: The use of mechanical couplers is frequently justified when space does not allow lapping)

3.3.1.1. Laps

(1) The detailing of laps between bars shall be such that:

i) The transmission of the forces from one bar to the next is assured;
ii) Spalling of the concrete in the neighborhood of the joints does not occur;
iii) Large cracks which affect the performance of the structure do not occur.

3.3.1.2. Location and position:

1) Between bars should normally be staggered and not located in areas of high moments/forces (e.g., plastic hinges.)
2) At any section should normally be arranged symmetrically.
3) Splice in flexural members should not be at sections where the bending moment is more than 50% of the moment of resistance of the section.

The arrangement of lapped bars should comply

1) The clear distance between lapped bars should not be greater than 4ϕ or 50 mm, otherwise the lap length should be increased by a length equal to the clear space where it exceeds 4ϕ or 50 mm;

2) The longitudinal distance between two adjacent laps should not be less than 0.3 times the lap length, l0;

3) In case of adjacent laps, the clear distance between adjacent bars should not be less than 2ϕ or 20 mm.

3.3.1.3. Lapping in Tension and compression:

By following above requirements, the permissible percentage of lapped bars in tension may be 100% where the bars are all in one layer. Where the bars in several layers the percentage should be reduced to 50%.

All bars in compression and secondary (distribution) reinforcement may be lapped in section.

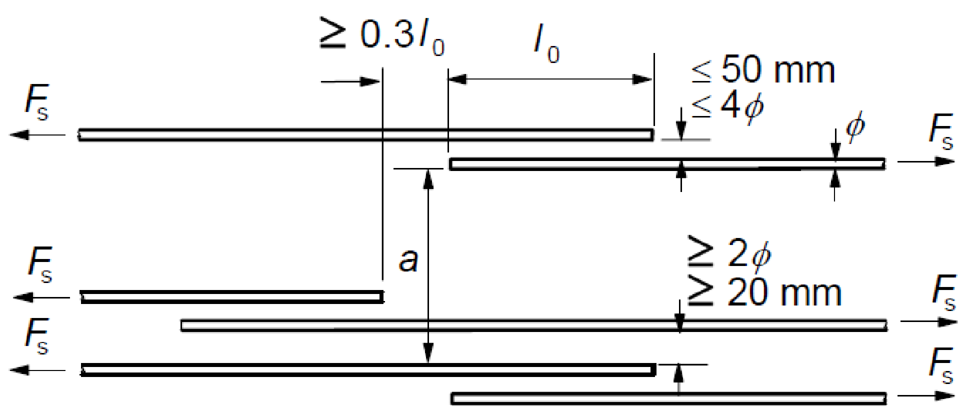


Fig-3.3.Adjacent laps.

3.3.1.4. Lap Length

(1) The design lap length is:

Where:

iscalculated from Expression

≥max {0.3; 15ϕ; 200 mm}

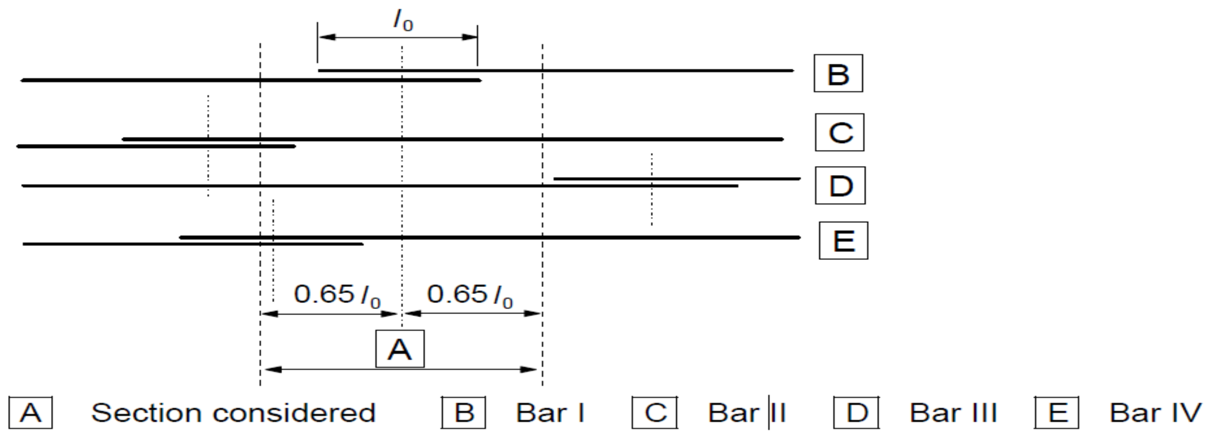
Values of(may be taken from Table 8.2, EBCS-1); however, for the calculation of α3, Σ should be taken as 1.0 with = area of one lapped bar.

= (0.5 but not exceeding 1.5 not less than 1.0, where is the percentage of reinforcement lapped within 0.65 from the centre of the lap length considered. Values of are given in Table 3.0.

Bars of different diameters: When bars of two different diameters are to be spliced the lap length shall be calculated on the basis of diameter of the small bar.

Percentage of lapped bars relative to the total cross-section area	**< 25%**	**33%**	**50%**	**> 50%**
	1	1.15	1.4	1.5
Note: Intermediate values may be determined by interpolation.				

Table3.0.Values of the coefficient



Example: Bars II and III are outside the section being considered: % = 50 and α6=1.4

Figure-3.4: Percentage of lapped bars in one lapped section

3.3.1.5. Transverse reinforcement in the lap zone:

1) Transverse reinforcement for bars in tension
2) Transverse reinforcement for bars permanently in compression.

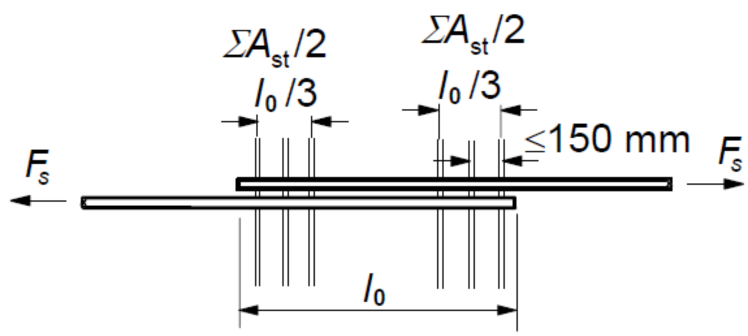
Diameter of lapped bars or percentage of	Transverse reinforcements bars in tension.

lapped bars in a section	
1) or percentage of lapped bars	Any transverse reinforcement or links necessary for other reasons may be assumed sufficient for the transverse tensile forces without further justification.
2)	, The transverse bars should be placed perpendicular to the direction of lapped reinforcement
	Transverse reinforcement total area(sum of all legs parallel to the layer of the spliced reinforcement) - Area of one lapped bar
3) If more than 50% reinforcement is lapped at one point and the distance "a" between adjacent laps at a section is	Transverse reinforcement should be formed by links or "U" bars anchored into the body of the section.

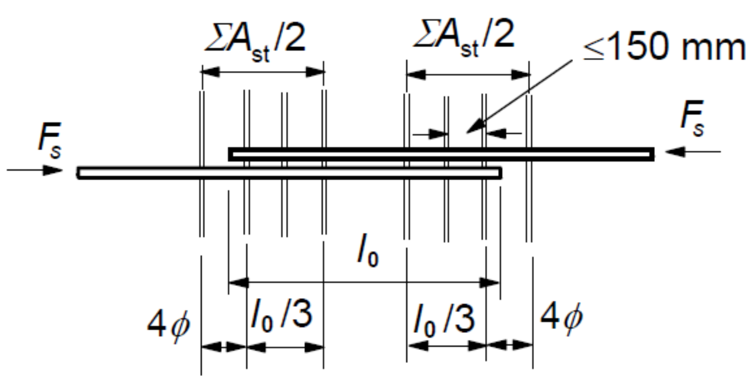
Table-3.1. Transverse reinforcement in the lap zone.

Transverse reinforcement for bars permanently in compression.

In addition to the rules for bars in tension one bar of the transverse reinforcement should be placed outside each end of the lap length and within 4 of the ends of the lap length.



(a) Bars in tension



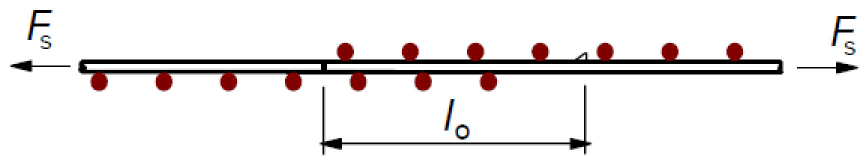
(b) Bars in compression

Fig-3.5. Transverse reinforcement for lapped splices.

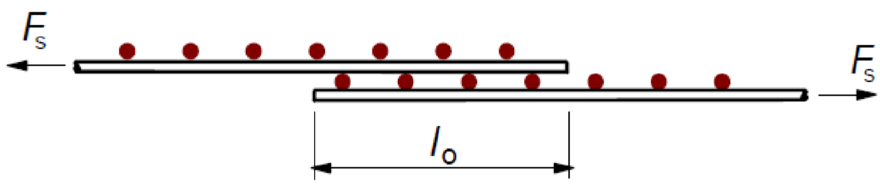
Laps for welded mesh fabrics made of ribbed wires

Laps of the main reinforcement:

1) Laps may be made by either by intermeshing or by layering of the fabrics.



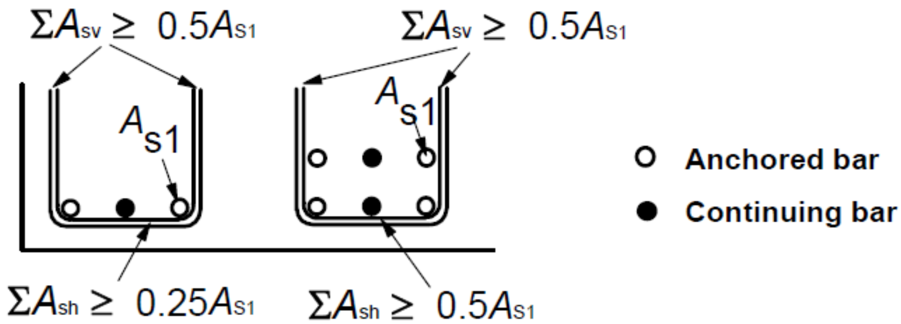
(a).Intermeshed fabric (longitudinal section)



(b).Layered fabric (longitudinal section)

Fig-3.6.lapping of welded fabric.

2) Where fatigue loads occur intermeshing should be adopted
3) The percentage of the main reinforcement which may be lapped in any one section should comply with the following
    a) For intermeshed fabric the values of is sufficient which is given above.
    b) For layered fabric the permissible percentage of the main reinforcement that may be spliced by lapping in any section depends on the specific cross section area of the welded fabric provided (S)Prov, where "s" is the spacing of wires.



In the left hand case n1 = 1, n2 = 2 and in the right hand case n1 = 2, n2 = 2

Figure-3.7.Additional reinforcement in an anchorage for large diameter bars where there is no transverse compression

3.4. Spacing of bars:

3.4.1. Clear distance:

The clear distance (horizontal and vertical) between individual parallel bars or horizontal layers of parallel bars should not be less than the maximum of , bar diameter(,is the maximum size of the aggregate(Clause 8.2)
from nation Annex- 1 and 5 respectively.

The spacing of bars shall be such that the concrete can placed and compacted satisfactorily for the development of adequate bond.

3.4.2.Spacers and chairs: The subject is not addressed in the EBCS-2 and Ec-2

Spacers and chairs are used to have a suitable concrete cover or to hold bars position. They are made of sundry Materials.

Spacers:

Mortar or galvanized or plastic or stainless steel wire elements designed to ensure a satisfactory concrete cover for reinforcing bars.

Chairs:

These may be discrete to support bars in a specific position or continuous for continuous supports. Chairs are usually made of galvanized or stainless steel wire.

Spacers are provided in slabs and footings for bottom reinforcement and for slab edges. On the other hand chairs and continuous chairs and special chairs for top reinforcement in slabs and footings.In beams spacers should be set in the stirrups at intervals of not more than 1000mm longitudinally. In columns along the length spacers should be placed in the stirrups not more than 100 times the minimum dia. of main reinforcement. And in walls spacers should be staggered in each layer of reinforcement at intervals measuring larger of 500mm or 50 times the dia. of the reinforcing bars.

Clause 7.3 BS 8110:

This clause of the code recommendations are given to ensure that the reinforcement is properly placed and the required cover is obtained. This is achieved during construction by inserting spacer blocks and chairs in the formwork on the reinforcement.

The spacer must be designed such that they are durable and will not lead to corrosion of the reinforcement or to palling of the concrete. The use of spacer blocks constructed on site from concrete is not permitted.

3.5. Permissible mandrel diameters for bent bars:

Where normal hooks are used they should be of U-type or L-type but usually U- type is preferred for mild steel bars and L-type for deformed bars. If the radius of the bend or hooks conforms to that of the standard hooks or bends in longitudinal bars the bearing stresses inside the bend in concrete need not be checked. Otherwise bearing stress must be checked by using the following formula.

Bearing stress =

Tensile force due to design loads in a bar or group of bars(n)

Internal radius of the bend(mm) and

Size of the bar or if in bundle the size of bar of equivalent area (mm)

For limit state method of design this stress shall not exceed

Where is the characteristic strength of the concrete

for a particular bar or group of bars in contact shall be taken as a c/c distance between bars or group of bars perpendicular to the plane of the bend (mm)for a bar or group of bars adjacent to the face of the member, "a" shall be taken as the cover plus size of bar.

In other words the minimum radius of the bend "r" should be such that

Where

The minimum diameter to which a bar is bent shall be such as to avoid bending cracks in the bar, and to avoid failure of concrete inside the bend of the bar and to avoid failure of concrete inside the bend of the bar.

In order to void damage to the reinforcement the diameter to which the bar is bent(Mandrel diameter) should not be less than

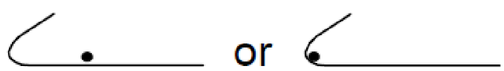
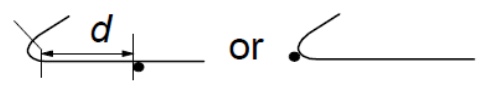
National Annex, below Table

a) For bars and wire

Bar diameter	**Minimum mandrel diameter for bends, hook and loops (see Figure 8.1)**

$\phi \leq 16$ mm	$4\ \phi$
$\phi > 16$ mm	$7\ \phi$

b) For welded reinforcement and mesh bent after welding

<table>
<tr><th colspan="2">Minimum mandrel diameter</th></tr>
<tr><td>
</td><td>
</td></tr>
<tr><td>5ø</td><td>$d \geq 3ø$: 5ϕ

$d < 3ø$ or welding within the curved zone: 20ø</td></tr>
<tr><td colspan="2">Note: The mandrel size for welding within the curved zone may be reduced to 5ø where the welding is carried out in accordance with EN ISO 17660</td></tr>
</table>

Table-3.2: Minimum mandrel diameter to avoid damage to reinforcement

The mandrel diameter need not be checked to avoid concrete failure if the following conditions exist:

i) either the anchorage of the bar does not require a length more than 5ϕ past the end of the bend or

ii) Thebar is not positioned at the edge (plane of bend close to concrete face) and there is a cross bar diameter ≥ ϕ inside the bend.”.

iii) The mandrel diameter is at least equal to the recommended values given in Table. Otherwise the mandrel diameter,,should be increased in accordance with below Expression

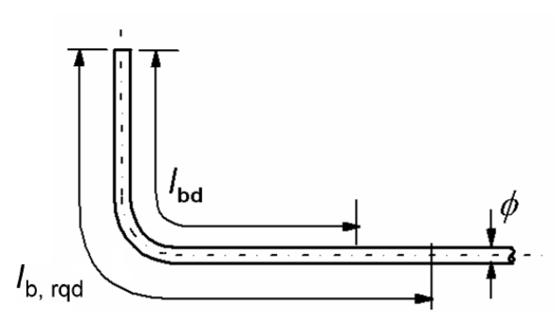
Where:

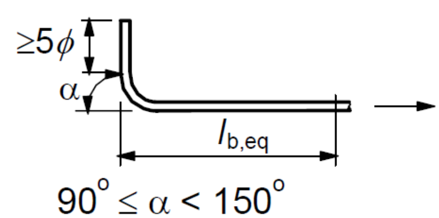
is the tensile force from ultimate loads in a bar or group of bars in contact at the start of a bend

for a given bar (or group of bars in contact) is half of the Centre-to-Centre distance between bars (or groups of bars) perpendicular to the plane of the bend. For a bar or group of bars adjacent to the face of the member,should be taken as the cover plus ϕ / 2.The value of should not be taken greater than that for concrete class C55/67.

3.6. Anchorage of longitudinal reinforcement:

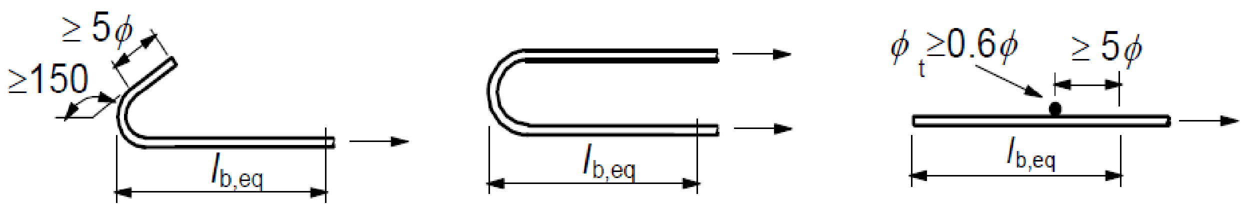
3.6.1. Reinforcing bars, wires or welded mesh fabrics: Reinforcing bars, wires or welded mesh fabrics shall be so anchored that the bond forces are safely transmitted to the concrete avoiding longitudinal cracking or sapling. Transverse reinforcement shall be provided if necessary.





a) Basic tension anchorage length $l_{b,rqd}$, standard for any shape measured along the center line

b) Equivalent anchorage length for bend



c) Equivalent anchorage length for standard hook

d) Equivalent anchorage length for standard loop

e) Equivalent length for welded transverse bar

Fig-3.8.Basic anchorage length

(1) The calculation of the required anchorage length shall take into consideration the type of steel

and bond properties of the bars.

(2) The basic required anchorage length, for anchoring the force, in a straight bar assuming constant bond stress equal to follows from:

Where is the design stress of the bar at the position from where the anchorage is measured from. Values for are given.

(3) For bent bars the basic required anchorage length, and the design length, , should be measured along the centre-line of the bar

(4) Where pairs of wires/bars form welded fabrics the diameter,ϕ, in the above Expression should be replaced by the equivalent diameter

3.6.2. Design anchorage length

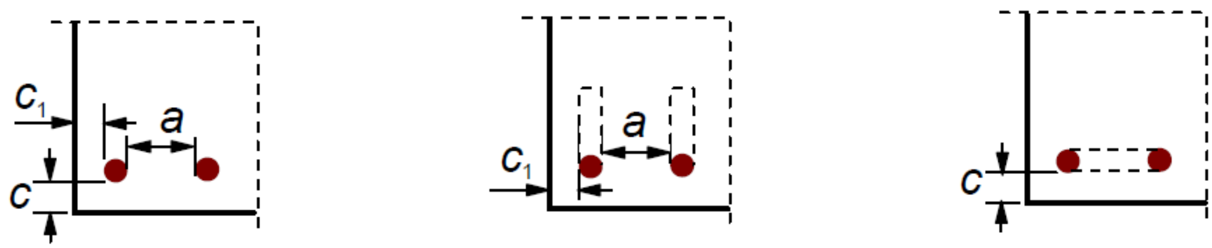
(1) The design anchorage, length, is:

(2) , = ≥

Where are coefficients given

is for the effect of the form of the bars assuming adequate cover

is for the effect of concrete minimum cover



a) Straight bars **b) Bent or hooked bars** **c) Looped bars**

cd = min (a/2, c1, c) cd = min (a/2, c1) cd = c

Fig-3.9: Values of Cd for beams and slabs

is for the effect of confinement by transverse reinforcement

is for the influence of one or more welded transverse bars ($\phi t > 0.6\phi$) along the design anchorage length l_{bd}

is for the effect of the pressure transverse to the plane of splitting along the design anchorage length

The product ($\alpha 2$ $\alpha 3$ $\alpha 5$) ≥ 0.7

is taken

is the minimum anchorage length if no other limitation is applied:

For anchorages in tension: ≥max {0.3; 10ϕ; 100 mm}

For anchorages in compression:≥max {0.6; 10ϕ; 100 mm}

$l_{b,rqd}$is calculated

Influencing factor	Type of anchorage	Reinforcement bar	
		In tension	In compression
Shape of bars	Straight	= 1.0	1.0
	Other than straight (see Figure 8.1 (b), (c) and (d) of EBCS-1)	= 0.7 if > 3ϕ Otherwise = 1.0 (see above Figure for values of)	= 1.0
Concrete cover	Straight	= 1 – 0.15 (- ϕ)/ϕ ≥ 0.7 ≤ 1.0	= 1.0
	Other than straight (see Figure 8.1 (b), (c) and (d) of EBCS-1)	= 1 – 0.15 (- 3ϕ)/ϕ ≥ 0.7 ≤ 1.0 (see above Figure for values of)	= 1.0
Confinement by transverse reinforcement not welded to main reinforcement	All types	= 1 – Kλ ≥ 0.7 ≤ 1.0	= 1.0
Confinement by welded transverse reinforcement*	All types, position and size as specified in Figure 8.1 (e) of EBCS-1)	= 0.7	= 0.7
Confinement	All types	= 1 – 0.04p	-

by transverse pressure		≥ 0.7 ≤ 1.0	
where: λ = (Σ - Σ)/ Σ cross-sectional area of the transverse reinforcement along the design anchorage length Σ cross-sectional area of the minimum transverse reinforcement = 0.25 As for beams and 0 for slabs area of a single anchored bar with maximum bar diameter K values shown in below Figure p transverse pressure [MPa] at ultimate limit state along			
For direct supports may be taken less than provided that there is at least one transverse wire welded within the support. This should be at least 15 mm from the face of the support.			

Table-3.3: Values of

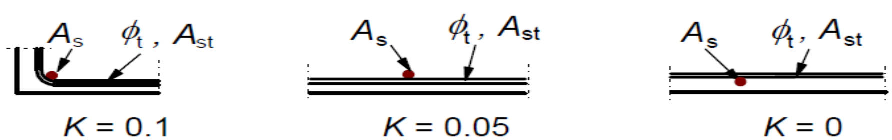


Fig- 3.10. Values of K for beams and slabs

3.6.3 Anchorage of links and shear reinforcement

(1) The anchorage of links and shear reinforcement should normally be effected by means of bends and hooks, or by welded transverse reinforcement. A bar should be provided inside a hook or bend.

(2) The anchorage should comply with belowFigures. Welding should be carried out in accordance with EN ISO 17660

Note: For definition of the bend angles see Figure3.11

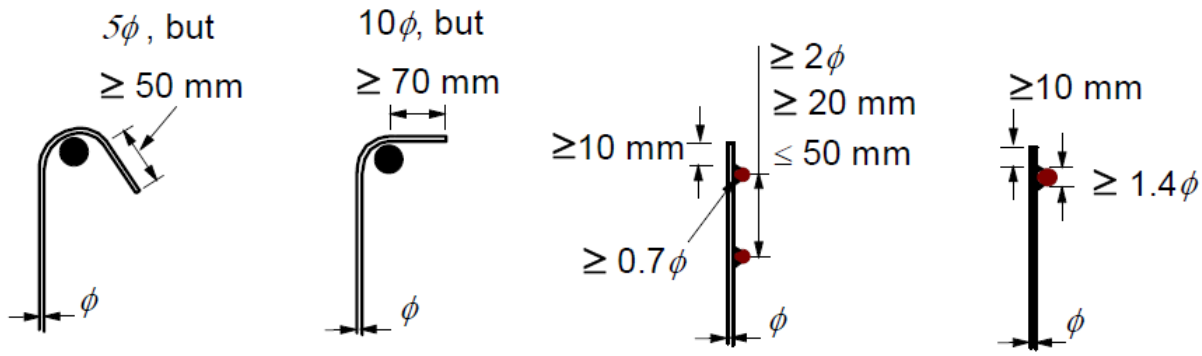


Fig 3.11-Anchorage of links

Anchorage by welded bars

Additional anchorage to that of the above may be obtained by transverse welded bars (see Figure 3.12) bearing on the concrete. The quality of the welded joints should be shown to be adequate

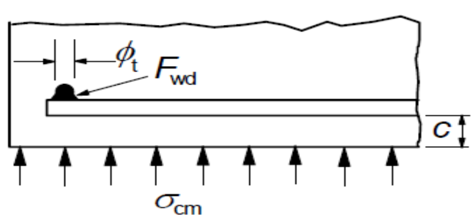


Fig-3.12. Welded transverse bar as anchoring device

(2) The anchorage capacity of one welded transverse bar (diameter 14 mm - 32 mm), welded on the inside of the main bar, is may then be reduced by /, where , is the area of the bar.

Note: The recommended value of is determined from:

= but not greater than

Where:

is the design shear strength of weld (specified as a factor times; say 0.5 where is the cross-section of the anchored bar and fyd is its design yield strength)

-- is the design length of transverse bar: ltd = 1.16 ϕt(fyd/σtd)0.5 ≤ lt

-- is the length of transverse bar, but not more than the spacing of bars to be anchored

is the diameter of transverse bar

is the concrete stress; σtd = (fctd + σcm)/y ≤ 3 fcd

is the compression in the concrete perpendicular to both bars (mean value, positive for compression)

y is a function: y = 0.015 + 0.14 e(-0.18x)

x is a function accounting for the geometry: x = 2 () + 1

c is the concrete cover perpendicular to both bars

Anchorage by BS 8110:

In many instances there is insufficient length available to incorporate an additional straight length bar and an alternative such as bend or a hook is used.

The most common locations for bends is at the simply supported ends of member where an effective anchorage length beyond the center line of the support equivalent to (12 * bar diameter) is required.

The effective anchorage length of a bend or hook is defined in Clause 3.12.8.23 as the greater of

1) 4r, where "r" internal radius of bend, 12 * bar diameter or the actual length of the bar (r- is equal to 3d in a standard hook or bend) for bends.
2) 8r, where "r" is the internal radius of the bend , 24* bar diameter or the actual length of bar(r- is normally assumed to be equal to "3d" in a standard hook or bend) for hooks

In addition any length of bar in excess of (4 * bar diameter) beyond the end of bend and which lies within the concrete in which the bar is to be anchored may also be included for effective anchorage.

l_{bd}b

llllbd

b

a) **Direct support**: Beam supported by wall or column

b) **Indirect support**: Beam intersecting another supporting beam

Anchorage length is measured from the line of contact between beam and support

Fig-3.13.Anchorage of bottom reinforcement at end supports

Where a beam is supported by a beam instead of wall or column reinforcement is provided and designed to resist the mutual reaction. The supporting reinforcement is in addition to that required for other reasons. The supporting reinforcement between two beams should consist of links surrounding the principal reinforcement of the supporting member. The supporting links may be placed in a zone beyond the intersection of beams.

Mechanical anchorages:

When length available for anchorage is small mechanical anchorages in the form of welded cross bars or end plates may be used. It is common in precast elements, corbels, brackets and at other support points.

Longer and stronger anchorage length of reinforcing bars in concrete to ensure failure by yielding prior to bond slippage as the latter failure is more brittle.

Ensure failure by yielding here instead of bond failure behind

Bar in tension

Longer and stronger anchorage

Fig-3.14.Longer bond and anchorage length of reinforcing bars.

.

Fig-(a) Radial force by bar inward on concrete which is relatively thick

Radial force by bar tending to cause concrete spalling if concrete is relatively thin

Fig-(b)

<u>Restraining and/or avoiding radial forces by reinforcing bars on concrete at where the bars change direction</u>

Fig3.15-Bars bent inwards to avoid radial forces on thin concrete cover.

3.7 .Surface reinforcement:

It may be necessary to provide surface reinforcement either to control cracking or to ensure adequate resistance to spalling of the cover.

Surface reinforcement to resist palling should be used where:

i) bars with diameter greater than 32 mm or

ii) bundled bars with equivalent diameter greater than 32 mm

The surface reinforcement should consist of wire mesh or small diameter bars, and be placed outside the links.

The area of surface reinforcement

>0.01 - in the direction perpendicular to large dia bars.

>0.02 -parallel to those bars.

Where the cover to reinforcement is greater than 70 mm, for enhanced durability similar surface reinforcement should be used, with an area of 0,005 $A_{ct,ext}$in each direction.

The longitudinal bars of the surface reinforcement may be taken into account as longitudinal bending reinforcement and the transverse bars as shear reinforcement provided that they meet the requirements for the arrangement and anchorage of these types of reinforcement

3.8. Bundled bars:

A bundle is defined as a group of parallel bars bundled in contact to act as unit. Not more than four bars can be grouped into one bundle.

Bundled bars can be used as column verticals.

In Beams When bars are placed in contact with each other in groups of 2, 3 or 4 as bundled bars. The minimumclear space provided between bundles for buildings under ACI 318(7.6.3) shall be equal to the diameter of a single round bar having an area equivalent to the area of the bundle. Unless otherwise stated the rules for individual bars apply to bundles of bars.

In a bundle all bars should of the same characteristics (type and grade)

Bars of different sizes may be bundled provided that the ratio of diameters does not exceed 1.7

In design the bundle is replaced by a notional bar having the same sectional area and the same center of gravity as the bundle. The equivalent diameters of this notional bar is such that.

is number of bars in bundle which is limited to for vertical bars in compression and for bars in a lapped joint.

for all other cases.

For a bundle the rules given above for individual bars apply for spacing.

The equivalent diameter " should be used, but the clear distance between bundles should be measured from the actual external contour of the bundle bars.

The concrete cover should be measured from the actual external contour of the bundles and should not be less than

Where two touching bars are positioned one above the other and where the bond conditions are good such bars need to be treated as a bundle.

Anchorage of bundles of bars:

Bundles of bars in tension may be curtailed over end and intermediate supports.

Equivalent diameter of bar < 32 mm May be curtailed near a support without the need for staggering bars

Equivalent diameter of bar mm.Which are anchored near a support, should be staggered in the longitudinal direction.

Where individual bars are anchored with a staggered distance greater than (Where based on the bar diameter) the diameter of the bar may be used in assessing .Otherwise the equivalent diameter of the bundle should be used.

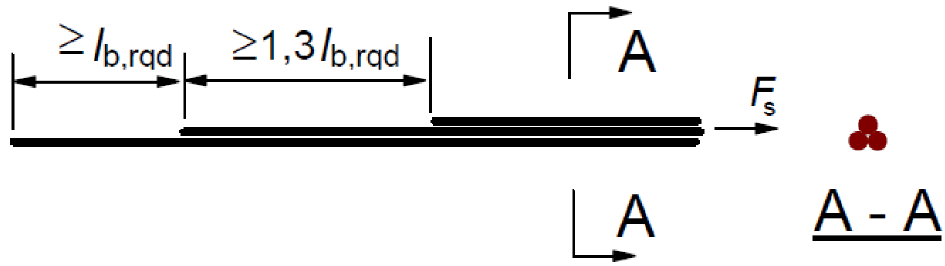


Fig-3.16: Anchorage of widely staggered bars in a bundle

For compression anchorages bundled bars need not be staggered.

For bundles with an equivalent dia. mm.At least four links having a diameter should be provided at the ends of the bundle .A further link should be provided just beyond the end of the curtailed bar.

Lapping bundles of bars:

The lap length should be calculated in accordance with the following formula using as the equivalent diameter of bar.

For bundles which consist of two bars

With an equivalent diameter is < 32 mm.The bars may be lapped without staggering individual bars. In this case equivalent bar size is used to calculate

For bundles which consist of two bars with

An equivalent diameter or of 3 bars.Individual bars should be staggered in the longitudinal direction by at least 1.3

(Where is based on single bar)

Care should be taken to ensure that there are not more than four bars in any lap cross section Bundles of more than 3-bars should not be lapped.

3.9. Concrete cover

Concrete cover is defined as it is the distance between surface of the reinforcement closest to the nearest concrete surface (including links and stirrups and surface reinforcement where relevant and the nearest concrete surface) (clause 4.4.1.1(1)).

The nominal cover shall be specified on the drawings it is defined as minimum cover plus an allowance in design for deviation (clause 4.4.1.1(2))

Minimum cover:-

Minimum concrete cover - shall be provided in order to ensure

i) The safe transmission of bond forces

ii) The protection of the steel against corrosion(durability)

iii) An adequate fire resistance

The greater value for " satisfying the requirements for both bond and environmental conditions shall be used.

Where

$c_{min,b}$ -- minimum cover due to bond requirement,

$c_{min,dur}$ -- minimum cover due to environmental conditions,

$\Delta c_{dur,\gamma}$ --additive safety element,

$\Delta c_{dur,st}$ -- reduction of minimum cover for use of stainless steel,

$\Delta c_{dur,add}$--reduction of minimum cover for use of additional protection,

In order to transmit bond forces safely and to ensure adequate compaction of the concrete the minimum cover should not be less than cmin,b. (Clause 4.4.1.2(3))

Bond Requirement	
Arrangement of bars	**Minimum cover $c_{min,b}$***
Separated	Diameter of bar
Bundled	Equivalent diameter (ϕ_n)
*: If the nominal maximum aggregate size is greater than 32 mm, $C_{min,b}$, should be increased by 5 mm.	

Table-3.4. minimum cover C min, b- requirements with regard to bond:

Structural Class

Criteria	Exposure Class according to Table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1	XD2/ XS1	XD3/ XS2/ XS3
Design working life of 100 years	Increase class by 2	Increase class by 2	Increase class by 2	Increase class by 2	Increase class by 2	Increase class by 2	Increase class by 2
Strength Class1)2)	C30/37 reduce class by 1	C30/37 reduce class by 1	C35/45 reduce class by 1	C40/50 reduce class by 1	C40/50 reduce class by 1	C40/50 reduce class by 1	C45/55 reduce class by 1
Member with slab geometry (Position of reinforcement not affected by construction process)	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1
Special Quality Control of the concrete production ensured	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1	reduce class by 1

Values of minimum cover $c_{min,dur}$requirements with regard to durability for reinforcement steel in accordance with EN10080.

Table-3.5.Recommended structural classification:

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

Table-3.6. Environmental requirement for

The concrete cover should be increased by additive safety element . The value of for use in a country may be found in its national Annex. The recommended value is Zero. "mm"(Clause 4.4.1.2(6))

Where stainless steel is used or where other special measures have been taken the minimum cover may be reduced by. For such situations the effects on all relevant material properties should be considered including bond.(clause 4.4.1.2(7))

Note: The recommended value .without further specification is “0” mm.
The concrete cover with additional protection (e.g. coating) the minimum cover may be reduced by. The recommended value for. Without further specification is “zero” mm (Clause 4.4.1.2(8))
<u>**In-situ concrete**</u> :Where in-situ concrete is placed against other concrete elements(precast or in situ) the minimum concrete cover of the reinforcement to the interface may be reduced to a value corresponding to the requirement for bond provided that(Clause 4.4.1.2(9))

i) The strength class of concrete is at least

ii) The exposure time of the concrete surface to an outdoor environment is short(< 28 days)

iii) The interface has roughened.

Allowance in design for deviation:

To calculate the nominal cover an addition to the minimum cover shall be made in design to allow for the deviation (.The required minimum cover shall be increased by the absolute value of the accepted negative deviation.(clause 4.4.1.3)

The value of for use in a country may be found in its national annex. The recommended value is 10 mm.

Nominal cover
(Specified in R.C drawing)

Minimum cover
(Durability and Bond)
Allowance for deviation

Axis distance, “a”
(Fire Protection)

Fig-3.17. For Specification of cover to reinforcement

Nominal cover (IS-456-2000):

It is the dimension used in design and drawings .It shall not be less than the diameter of the bar. For a longitudinal reinforcing bar in compression in a column nominal cover shall in any case not be less than 40mm or less than the diameter of such bar. In the case of columns of minimum dimension of 200 mm or under whose reinforcing bars do not exceed 12 mm a nominal cover of 25 mm may be used.

For footings minimum cover shall be 50 mm

Clause 2.2.4 and 3.1.5 of BS 8110:

The integrity of the reinforced concrete depends on its ability to prevent corrosion of the reinforcement when exposed to a wide range of environmental conditions. In addition to protection against corrosion of the steel, fire resistance requirements are also necessary to allow sufficient time to evacuate a building an prevent premature failure, spalling of the concrete must be avoided and adequate bond forces must develop between the reinforcement and concrete.

The essential elements of design which ensure adequate durability are the structural form/ detailing and the amount of concrete cover provided to protect the steel.

The actual concrete cover provided varies due to a number of factors such as:

1) Construction tolerances inherent in buildings the formwork(i.e., the mould into which the concrete is cast)
2) Variations in dimensions of the reinforcement resulting from the cutting bending of the steel and
3) Errors occurring during the fixing of the steel in the formwork.

There area number of criteria to be considered when determining the nominal cover. They are

1) Bar size: (Clause 3.3.1.2 BS 8110):
    a) Single bar: nominal cover main bar diameter

b) Paired bars: nominal cover main bar

c) Bundled bars: nominal cover

Where is the cross sectional area equal to the sum of the cross sectional area of the bars in bundle, for example if 4 bars in bundle , where "d" diameter of one bar.

2) Nominal maximum aggregate size: (Clause 3.3.1.3 of BS 8110):

Nominal cover nominal maximum size of aggregate i.e, normally

In most cases 20 mm size aggregate is suitable.

3) Uneven surfaces (Clause 3.3.1.4 BS 8110):

When concrete is cast on uneven surfaces (e.g., earth or blinding which is finely crushed aggregate rolled on the top of compacted fill such as Hard core) additional cover specified to that indicated by this code should be provide as shown below.

Concrete

Concrete **Nominal cover**

Uneven earth surface

Fig-3.18 Concrete cast directly on the earth – nominal cover from average soil level

Concrete

ConcreteNominal cover

Blinding layer

Hard Core

Fig- 3.19 Concrete cast on an adequate blinding layer(e. g: 50 mm thick)

<u>For uneven surfaces</u> :(e.g. exposed aggregate) the minimum cover should be increased by at least 5mm.(EBCS clause 4.4.1.2(11))

4) Ends of straight bars (Clause 3.3.2 BS 8110)
   Normally 40 mm cover is provided at the ends of straight bars , however as indicated in this clause where the end of a floor or roof unit is not exposed to the weather r to condensation cover is not mandatory.

Minimum dimensions (Clause 3.3.36 of BS 8110):

In addition to nominal cover requirements the code also specifies minimum dimensions (i.e, beam widths, rib widths, floor and wall thicknesses and column widths) for some structural elements to provide adequate fire resistance. The dimensions are given to ensure minimum periods of fire resistance ranging from 0.5 hours to 4 hours and elate specifically to the covers given in the BS 8110 Table 3.4.

Chapter-4

Structural Elements

4.1. BEAMS

Criteria	**EBCS-2,Final Draft**	**EC-2, values**	**BS 8110,values**	**IS 456-2000**
Main Reinforcement:				
Minimum area of main bars in tension	Clause 9.2.1.1(1) 0.26	0.26	0.0013bh	
Maximum area of tension reinforcement	Clause 9.2.1.1(3) 0.04	0.04 bd	0.04 bh	0.04bD
Main bars in compression	--------	-----------	0.002bh	0.04bD
Spacing of main bars,	--------			
Links				
Minimum area of	-----------		0.4bs/0.87	

shear reinforcement				
Spacing	---------	0.75d	0.75d	0.75d for vertical stirrups and “d” for inclined stirrups.
	----------	0.75d	d or 150mm from main bar	Shall not be greater than 300 mm

Table 4.1. Beam values

4.2. Other code provisions:

IS 456 -2000:

Where the maximum shear stress calculated is less than half the permissible value and in members of minor structural importance such as lintels this provision (minimum shear reinforcement) need not be complied with.

Clause 26.5.1.3(IS 456-2000):

Apart the specified reinforcement in the table if the depth of the web in a beam exceeds 750 mm side face reinforcement shall be provided along the two faces. The total area of such reinforcement shall be not less than 0.1% of the web area and shall be distributed equally on two faces at a spacing not exceeding 300 mm or web thickness whichever is less.

Clause 3.12.5.4 BS 8110:

In deep beams exceeding 750 mm overall depth there is a risk of local yielding of bars in the side faces which can lead to crack in the web. To control cracking in this situation longitudinal bars should be distributed at spacing 250 mm near the faces of the beam distributed over a distance of   of the beam depth measured from the tension face.

The minimum size of the bar to be used in the side face is , bar diameter

is the bar spacing

Breadth of the section

Fire resistance requirements for beams: (Clause 3.3.6 BS 8110):

The fire resistance of a beam depends on its width and the concrete cover. Note that in practice the fire resistance requirements may dictate the size of beam and the concrete cover.

Clause 4.3.1, part-2, BS 8110:

Fire rating (Hours)	Minimum width (mm)		Concrete cover to main reinforcement (mm)	
	Simply supported	Continuous	Simply supported	Continuous
1	120	80	30	20
2	200	150	50	50
3	240	200	70	60
4	280	240	80	70

Table 4.1 Fire resistance requirements for beams.

Slenderness limits: (Clause 3.4.1.6 BS 8110):

To guard against lateral buckling the slenderness limits should not be exceeded as given below table. The term slenderness limit is defined as the clear distance between lateral restraints “d” is the effective depth and “ the breadth of the compression face of the beam midway between restraints.

Slenderness limits:

Type of beam	Slenderness limits
Simply supported	
Continuous	Same as above
Cantilever	

Table 4.2. Slenderness limit for beams

4.3. EBCS provisions:

4.3.1. Elements of the structure :(clause 5.3.1)

1) The elements of a structure are classified, by consideration of their nature and function, as beams, columns, slabs, walls, plates, arches, shells etc. Rules are provided for the analysis of the commoner of these elements and of structures consisting of combinations of these elements.
2) A beam is a member for which the span is not less than 3 times the overall section depth. Otherwise it should be considered as a deep beam

4.3.2. Reinforcement:

4.3.2.1. Minimum reinforcement

Minimum areas of reinforcement are given in order to prevent brittle failure, wide cracks and also to resist forces arising from restrained actions.

The area of longitudinal tension reinforcement should not be taken as less than

Where denotes the mean width of tension zone,

For a T-beam with the flange in only the width of the web is taken into account in calculating the value of

Should be determined with respect to the relevant strength class (given in table 3.1 of EBCS-2)

Alternatively for secondary elements where some risk of brittle failure may be accepted may be taken as 1.2 times the area required in ULS verification.

Sections containing less reinforcement than, should be considered as unreinforced.

The cross sectional area of tension or compression reinforcement should not exceed outside lap locations.

Strength class	C	C16/20	C20/25	C25/30	C30/35	C35/45	C40/45	C45/50	C50/60	C28/35	C32/40
	0.130	0.130	0.130	0.135	0.151	0.166	0.182	0.198	0.213	0.146	0.156

Table-4.3. Minimum percentage of steel

4.3.2.2. Design of supports:

In monolithic construction even when simple supports have been assumed in design the section at supports should be designed for a bending moment arising from partial fixity of at least of the maximum bending moment in span.

for beams= 0.15(National Annex)

4.3.3. GEOMETRIC DATA

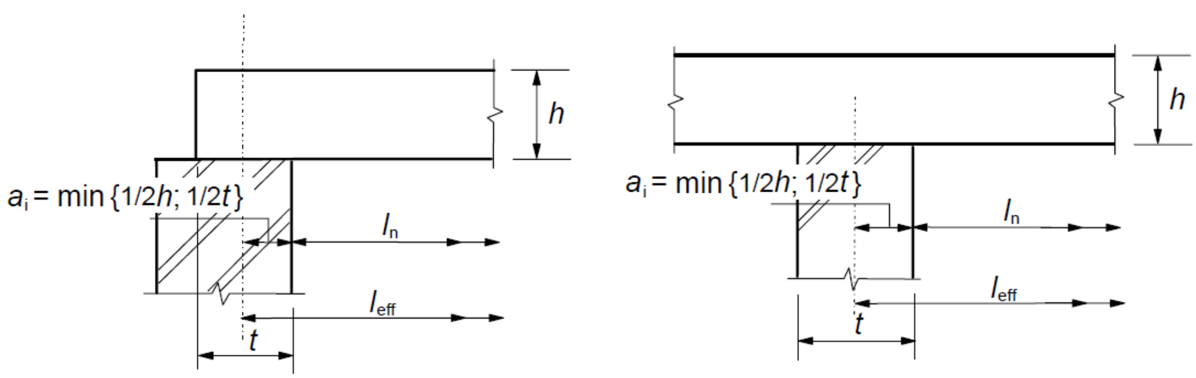
4.3.3.1 .Effective spans of beams and slabs in building (Clause 5.3.2.2)

The effective span, l_{eff} of a member should be calculated as follows:

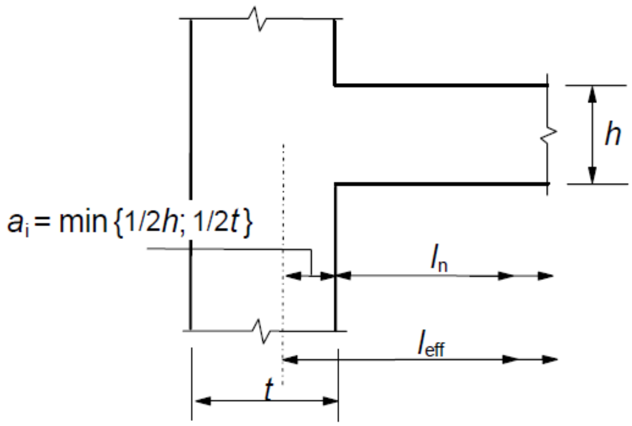
Where:

l_n is the clear distance between the faces of the supports;

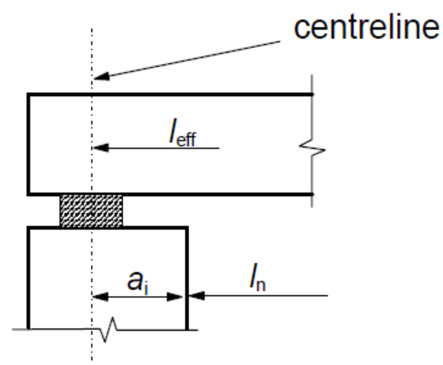
Values for a_1 and a_2, at each end of the span, may be determined from the appropriate a_i values in Figure given below where"t"is the width of the supporting element as shown.



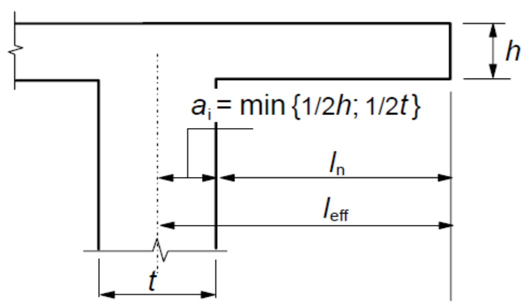
a) Non continuous members b) Continuous members



a) Supports considered fully restrained



d) Bearing provided



e) Cantilever

Fig- 4.0.Effective span for different support conditions.

<u>No restraint to rotation support</u>:

i) Assumption: Continues slabs and beams may generally be analyzed that the supports provide no rotational restraint.(e.g., over walls).

ii) A beam or slab is continuous over this type of support, the design support moment, calculated on the basis of a span equal to the centre-to centre distance between supports, may be reduced by an amount as follows:

-

t- is the breadth of the support

Note: Where support bearings are used t should be taken as the bearing width

<u>Monolithic support for beam or slab:</u>

i) For this condition the critical design moment at the support should be taken at the face of the support

ii) The design moment and reaction transferred to the supporting element (e.g. column, wall etc.) should be generally taken greater of the elastic or redistributed values.

Note: The moment at the face of the support should not be less than 0.65 that of the full fixed end moment.

4.3.3.2. Effective width of flanges (for all limit states)

1) The effective width of flange should be based on the length between points of aero moment, which may be obtained from the below figure.

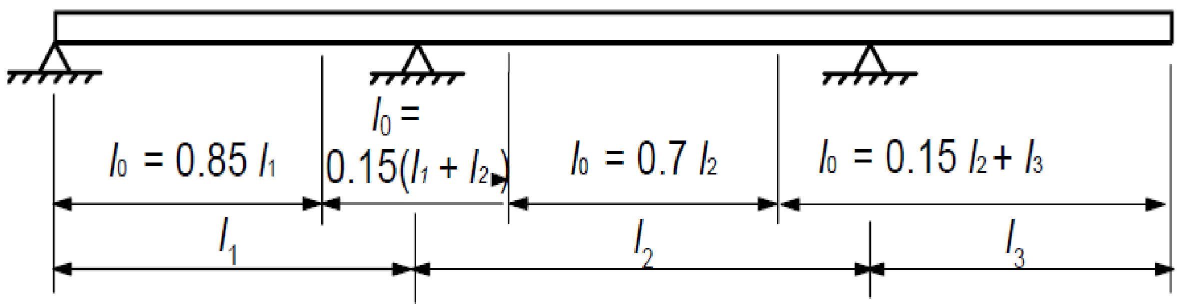


Fig-4.1: Definition of", for calculation of effective flange width

Note: The length of the cantilever, l_3, should be less than half the adjacent span and the ratio of adjacent spans should lie between 2/3 and 1.5.

2) In T beams the effective flange width, over which uniform conditions of stress can be assumed, which depends on the web and flange dimensions, the type of loading, the span, the support conditions and the transverse reinforcement

3) The effective flange width b_{eff} for a T beam of L beam maybe derived as:

Where

and

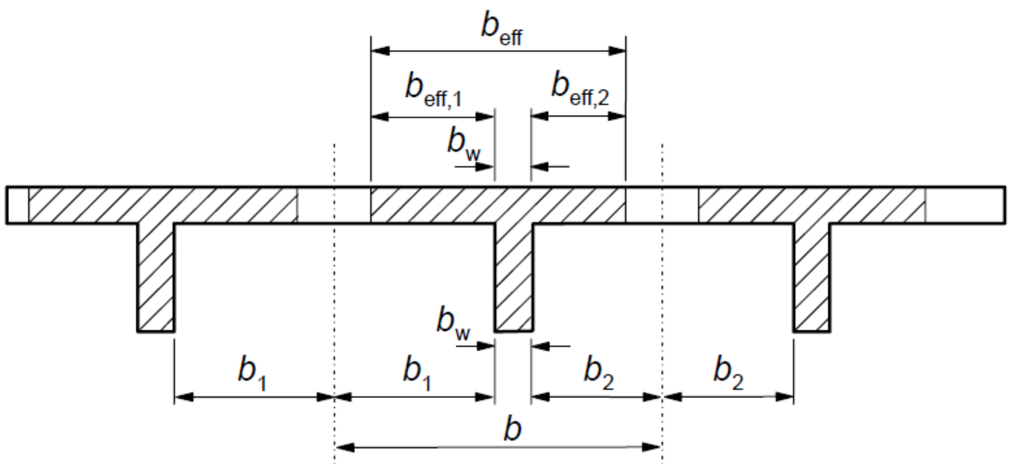


Fig-4.2 Effective flange width parameters

4.3.4. Transverse Reinforcement:

Transverse reinforcement in concrete with stronger anchorage by means of hooks with bent angles for ensuring better performance of the transverse reinforcements.

Comparison of anchorage between bend and

i) The end of the bend is anchored into the interior of the concrete member and thus risk of "opening up" is reduced.

ii) bend can easily open up and spall the concrete cover under tensile load

a) **b)**

Fig-4.3, Transverse reinforcement anchorage bends

As discussed by Law and Mak (2013) though the hook is unarguably better option, the hook is more popular as it has the relative ease of placement. The hook is much more difficult to place especially when the cast in bars are misaligned. However if there are other physical restraints such as adjoining beams or slabs preventing the opening up of the hook, the use of the hook should be acceptable.

Stirrup anchorage: (As3600-2009- Clause 8.2.12.3)

Stirrup anchorages should be located in the compression zone and be shown on structural drawings.

The area of shear reinforcement required at a particular cross section should be provided for a distance "D" from that cross section in the direction of decreasing shear

4.4. T-Beams:

4.4.1. Placing of tension reinforcement in flanged cross section:

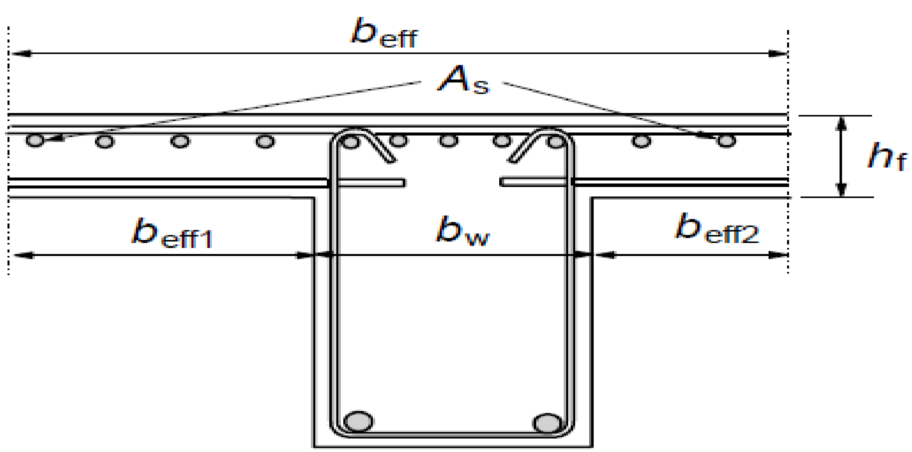


Fig-4.4.Distribution of tension reinforcement in flanged cross section

Tension reinforcement in a flanged beam at supports should be spread over the effective width. Any compression longitudinal reinforcement (diameter which is included in the resistance calculation should be held by transverse reinforcement with spacing not greater than "15."

Reinforcement in flanges of T and L beams (IS 26.5.1.8)

Where the flanges in tension a part of the main reinforcement shall be distributed over the effective flange width or a width equal to one tenth of the span. Whichever is smaller. If the

effective flange width exceeds one tenth of the span nominal longitudinal reinforcement shall be provided in the outer portions of the flange.

4.4.2. Curtailment of longitudinal reinforcement:

Sufficient reinforcement should be provided at all sections to resist the envelope of the acting tensile force, including the effect of inclined cracks in webs and flanges. For members with shear reinforcement the additional tensile force should be calculated by

hereis the maximum moment along the beam.

-Applied shear force

Angle between shear reinforcement and beam axis

Angle between the concrete compression strut and the beam axis.

Inner lever arm –in the shear analysis of reinforced concrete without axial force the approximate value may normally be used

For members without shear reinforcement may be estimated by shifting the moment curve a distance

The “shift rule “may also be used as an alternative for members with shear reinforcement where

Curtailment of tension reinforcement in flexural members (IS 456-2000)

For curtailment reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of the member or 12 times the bar diameter which ever greater except at simple support or end of cantilever.

Note: a point at which reinforcement is no longer required to resist flexure is where the resistance moment of the section considering only the continuing bars is equal to the design moment

Curtailment of bundled bars: (IS456-2000, clause-26.2.3.5)

Bars in bundle shall terminate at different points spaced apart by not less than 40 times the bar diameter except for bundles stopping at a support.

Positive moment reinforcement (IS 456-2000, Clause- 26.2.3.3)

At least one third the positive moment reinforcement in simple members and one fourth the positive reinforcement in continuous members shall extend along the same face of the members into the support to a length equal to

Negative moment reinforcement:

At least one third of the total reinforcement provided for negative moment at the support shall extend beyond the point of inflection for a distance not less than the effective depth of the member of or of the clear span whichever is greater.

4.4.3. Anchorage of bottom reinforcement:

The anchorage length of a bent up bar which contributes to the resistance to shear should be not less than in the tension zone and   in the compression zone , it is measured from the point of intersection of the axes of the bent up bar and the longitudinal reinforcement.

4.4.3.1. At end supports with little or no end fixity:

bottom steel at support provided in the span (National Annex),is required from the line of contact at the support.

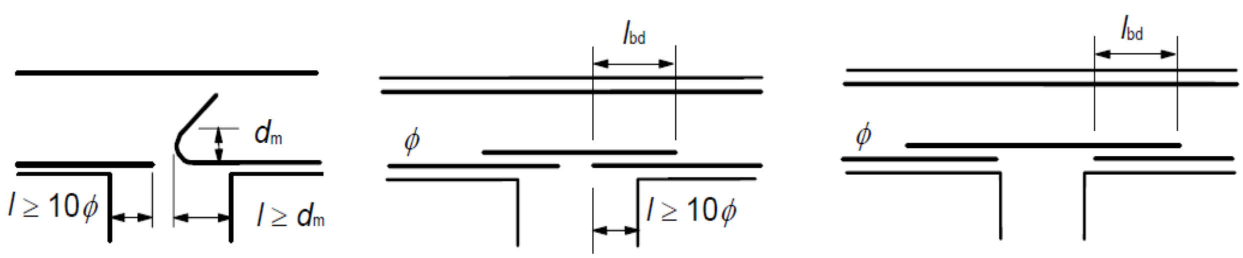
In case of simple support, the support conditions are may be direct or indirect. Direct support is such when beam is supported by wall or column and indirect support is when beam intersecting another supporting beam. In case of direct support transverse pressure may only be taken into account.

On the other hand indirect support, where a beam is supported by a beam instead of wall or column, reinforcement should be provided and designed to resist the mutual reaction .The supporting reinforcement is in addition to that required for other reasons. The supporting reinforcement between two beams should consist links surrounding the principal reinforcement of the supporting member. The supporting links may be placed in a zone beyond the intersection of beams.

4.4.3.2. Anchorage of bottom reinforcement at intermediate supports:

1) The anchorage length > 10(for straight bars) or diameter of mandrel (for hooks and bends with bar diameters at least equal to 16mm) or twice the diameter of the mandrel(in other cases.) (Figure(a))

2) The reinforcement required to resist possible positive moments (e.g. settlement of the support, explosion, etc.).This reinforcement should be continuous which may be achieved by means of lapped bars (see Figure (b) or (c)).



(a) (b) (c)

4.5. Anchorage at intermediate supports

Bends and hooks (IS-456-2000)

The anchorage value of the bend shall be taken as 4 times the diameter of the bar for each bend subject to a maximum of 16 times the diameter of the bar.

The anchorage value of a standard “U-type” hook shall be equal to 16 times the bar dia.

Mechanical devices for anchoraging:

Any mechanical or other device capable of developing the strength of the bar without damage to concrete may be used as anchorage with the approval of the engineer in charge.

Anchoraging bars in compression (IS-456-2000)

The anchorage length of straight bar in compression shall be equal to the development length of the bars in the compression as

Nominal diameter of the bar

Stress in bar at the section considered at design load and

Design bond stress.

Development in compression (IS-456-2000):

The projected length of hooks, bends and straight lengths beyond bends if provided for a bar in compression shall only considered for development length

Development length in Tension:

The development length includes anchorage values of hooks in tension reinforcement

Anchoraging shear reinforcement (IS 456-2000):

A) Inclined bars:- The development length shall be as for bars in tension , this length shall be measured under

1) In tension zone rom the end of the sloping or inclined portion of the bar and

2) In the compression zone from the mid depth of the section

B) Stirrups:- In case of secondary reinforcement – such as stirrups and transverse ties , complete development lengths and anchorage shall be deemed to have been provided when the bar is bent through an angle of at least round a bar of at least it's own diameter and is continued beyond the end of the curve for a length of at least eight diameter or When the bar is bent through the angle of and is continued beyond the end of the curve for a length at least six bar diameters or when the bar is bent through an angel of and is continued beyond the end of the curve for a length of at least four diameters.

Simplified rules for curtailment of bars: (Clause 3.12.10.2 BS 8110):

In practical design bending moment diagram are often not drawn for members of secondary importance and the theoretical cutoff points are not then known without further calculation. BS 8110 permits the simplified rules to be applied where beam supports substantially uniformly distributed loads. In these rules " refers to the effective span length and the bar size.

Simply supported beams: All the tension bars should be extended to within of the centers of the supports. At least 50% of these bars should further extend for at least (or its equivalent in hooks or bends) beyond centers of supports.

Cantilevers: All the tension bars at the support should extend a distance of whichever is greater. At least 50% of these bars should extend to the end of the cantilever.

Continuous beams of approximately equal spans:

1) All the tension bars at the support should extend from the face of the support, whichever is greater. At least 60% of these bars should extend and at least 20% should continue through the spans

2) All the tension bars at mid span should extend to within o.15*l* of interior supports and 0.1*l* of the exterior supports. At least 30% of these bars should extend to the centerof the supports.

4.4.4. Simplified detailing rules for beams:

a) Continuous member top reinforcement

Reinforcement for maximum hogging moment

100%

60%

b) Continuous member bottom reinforcement

30%

100%

Position of effective support *Reinforcement for maximum sagging moment*

c) Simplesupport bottom reinforcement

25%

Position of effective support100%

Where, effective length

is the distance to allow for tensile force due to shear force

-design anchorage length,

:Minimum of two spans required, Applies to U.D.L only for above fig.'s

Fig-4.6. Curtailment of reinforcement for various supports

4.5. Shear reinforcement:-

The shear reinforcement should form an angle "of between 45^0 and 90^0 to the longitudinal axis of the structural element.

The shear reinforcement may consists combination of

1) Links enclosing the longitudinal tension reinforcement and the compression zone
2) Bent up bars
3) Cages, ladders etc. which are cast in without enclosing the longitudinal reinforcement but are properly anchored in the compression and tension zones.

At least of the necessary shear reinforcement should be in the form of links.(National Annex)

A

- Inner link alternatives

B

-Enclosing link

Fig-4.7.Shear reinforcement

The ratio of shear reinforcementshear reinforcement ratio >

Area of shear reinforcement within length “s”

s- is the spacing of shear reinforcement measured along the longitudinal axis of the member.

bredth of the web of the member

angle between shear reinforcement and the longitudinal axis

For beams=

The maximum longitudinal spacing between shear assemblies <;

The maximum spacing of bent up bars <;

The transverse spacing of the legs in a series of shear links <

4.6. Torsion reinforcement:

The torsion links should be closed and be anchored by means of laps or hooked ends and should form an angle of 90^0 with the axis of the structural element

The longitudinal spacing of the torsion links:-

< U/8 (U- outer circumference of the cross section) or

<0.75d (1+cot) or the less dimension of beam cross section.

Fig-4.8.Recommended shapes for torsion reinforcement

The longitudinal bars:-

1) At least one bar at each corner
2) Others distributed uniformly around the inner periphery of the links spacing < 350mm

Surface reinforcement:

Either to control cracking or to ensure adequate resistance to spelling of the cover (Annex J, EBCS)

4.7. Deep beams:

A beam is a member for which the span is not less than 3 times the overall section depth, otherwise it should be considered as a deep beam (Clause 5.3.1(3))

Deep beams should normally be provided with an orthogonal reinforcement mesh near each face with a minimum of on each face and each direction

The distance between two adjacent bars of the mesh < Min of

Chapter-5

Slabs:

5.1. Introduction

Definition: A slab is a member for which the minimum panel dimension is not less than 5 times the overall slab thickness(Clause 5.3.1(4))

Slab may be supported on two sides and having the bending predominantly in one direction only called one way slab and the slab supported on four sides and having the bending in both sides called two slabs. And there is a difference in steel provision in the one way and two way. In both slabs steel is provided in both X and Y directions but in one way the steel parallel to short direction provided is main reinforcement and in the other direction providing steel is called secondary or distribution steel. But in two slabs the steel provided in the both directions is main reinforcement.

Slabs:

Criteria	EBCS-2	EC-2	BS-8110	IS 456-2000
Main bars in Tension:				
	Clause 9.3.1.1(1) 0.26	0.26	0.0013 bh	Slab is usually designed as beam of one meter width to carry moment over a strip of “1” meter
	Clause 9.3.1.1(1) 0.04	0.04bd	0.04bh	Slab is usually designed as beam of one meter width to carry moment over a

				strip of “1” meter
Secondary transverse bars				
	Clause 9.3.1.1(2) 0.2 for one way slabs	0.2for single way slabs	0.002bh	0.15% of total cross section if mild steel is used. And 0.12% of total cross section if is used.
	-----------------	0.04bd	0.04bh	----------
Spacing of bars:				
	------------------			--------
	Clause 9.3.1.1(3) Principle reinforcement 3h400mm Secondary reinforcement 3.5h, Areas of maximum moment or areas of concentrated loads	Main 3h400mm Secondary reinforcement 3.5h Places of maximum moment Main	3d or 750mm	3D or 300 mm (whichever is smaller)

	Principal 2h Secondary	2h Secondary		
Punching shear:				
Links	Clause 9.4.3(2)	Link leg = 0.053	Total = 0.4ud/0.87	For slabs nominal shear stress shall not exceed. Shear reinforcements in slabs should be avoided since they workout cumbersome andexpensive. Hence, if increase the thickness of slab and redesign.
Spacing of links:				
	Clause 9.4.3(1) 0.75d	0.75d	0.75d	-------

	Clause 9.4.3(1) Within 1st control perimeter= 1.5d Outside 1st control perimeter= 2d	Within 1st control perimeter= 1.5d Outside1st control perimeter= 2d	1.5d	--------

Table 5.0 slabs values

Rectangular slab: (BS 8110):

Situation	Definition Percentage	Minimum percentage	
Rectangular section (in solid slabs this minimum should be provided in both directions)		0.24	0.13

Note: The minimum area of reinforcement is required in both directions in a slab.

Table 5.1.Minimum percentage of reinforcement

Minimum spacing of bars (**BS 8110)**: Guidance is given in the code for minimum bar spacing to ensure that members can be constructed achieving adequate penetration and compaction of the concrete to enable the reinforcement to perform as designed.

Maximum spacing bars: (Clause 3.12.11.2 BS 8110): The requirement **to** limit the maximum spacing of reinforcement is to minimize surface cracking.

5.2 .Flexural reinforcement:

For minimum and maximum steel percentages in the main direction – minimum and maximum areas as beams i.e.

Secondary transverse reinforcement > 20% of the principal reinforcement (main reinforcement) in one way slabs.

Areas near supports transverse reinforcement to principal top bars is not necessary where there is no transverse bending moment.

Minimum reinforcement (IS-456-2000):

The mild steel reinforcement in either direction in slabs shall not be less than 0.15% of the cross sectional area and in case of high deformed bars or welded wire fabric are used is 0.12%.

Maximum diameter bar in the slab: (IS-456-2000):

The diameter of reinforcing bars shall not exceed one eight of the total thickness of the slab.

5.2.1. Spacing of bars:

<, slabs.

For the principle reinforcement 3h400mm

For secondary reinforcement 3.5h 450mm

h-is the total depth of the slab.

In areas with concentrated loads or areas of maximum moment those provisions become respectively

The principle reinforcement 2h

For secondary reinforcement 3h400mm

Curtailment: As beams except for the “shift” rule may be used.

Where partial fixity exists, not taken into design:

Internal supports:of in adjacent span.

End supports:of in adjacent span

This top reinforcement should extend0.2 adjacent spans.

Spacing of reinforcement (IS_456-2000):

The horizontal distance between parallel reinforcement bars provided against shrinkage and temperature shall not be more than 5 times the effective depth of a solid slab or 450 mm whichever is smaller.

5.2.2.Corner reinforcement:

If the detailing arrangements at a support are such that lifting of the slab at a corner is restrained; suitable reinforcement should be provided.

Reinforcement at free edges of slab (clause 9.3.1.4)

Along a free (unsupported) edge a slab should normally contain longitudinal and transverse reinforcement generally arranged as showed in the below figure.

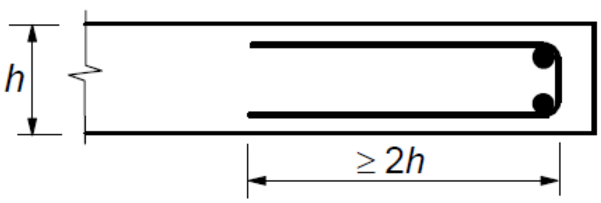


Fig-5.0.The normal reinforcement provided for a slab may act as edge reinforcement.

5.3. Shear reinforcement:

For shear reinforcement slab depth 200mm

In slabs if if shear reinforcement may consist entirely of bent up bars or of shear reinforcement assemblies.

The maximum longitudinal spacing of successive series of links is given by

Where “ is the inclination of the shear reinforcement

The maximum longitudinal spacing of bent up bars is given by

The maximum transverse spacing of shear reinforcement should not exceed 1.5d

5.4. Flat slabs:

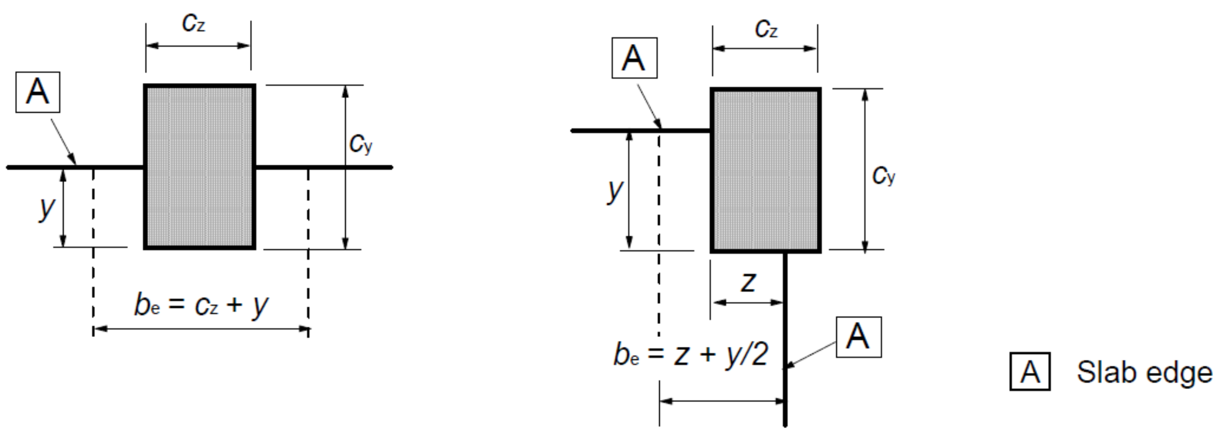
Support conditions of the slab determine the detailing of slab. The slab may be supported on columns or walls or beams. Therefore slab supported directly by columns are called flat slabs

Arrangement of top reinforcement should reflect behavior under working conditions.

At internal columns for reinforcement of area 0.5 should be placed in a width =0.25 panel width.

At least two bottom bars should pass through internal columns in each orthogonal direction.

Design reinforcement at edge and corner reinforcement should be placed within the "b_e"



Note: y can be >cy

Note: z can be >cz and y can be > cy

a) Edge column **b) Corner column**

Note: y is the distance from the edge of the slab to the innermost face of the column.Effective width "b_e" of flat slab

Fig-5.1.Edge and corner column reinforcement

5.4.1. Punching shear reinforcement:

Punching shear does not use the variable strutinclinate method and is similar to BS 8110 methods.

Where punching shear reinforcement is required it should be placed between the loaded area/column and kd inside the control perimeter at which shear reinforcement is no longer required. It should be provided in at least two perimeters of link legs. The spacing of the link leg perimeters should not exceed 0.75d.

The spacing of link legs around a perimeter should not exceed 1.5d within the first control perimeter (2d from loaded area), and should not exceed 2d for perimeters outside the first control perimeter where that part of the perimeter is assumed to contribute to the shear capacity.

For bent down bars as arranged in Figure one perimeter of link legs may be considered sufficient

Bent up bars passing through the loaded area or at a distance not exceeding 0.25d from this area may be used as punching shear reinforcement.

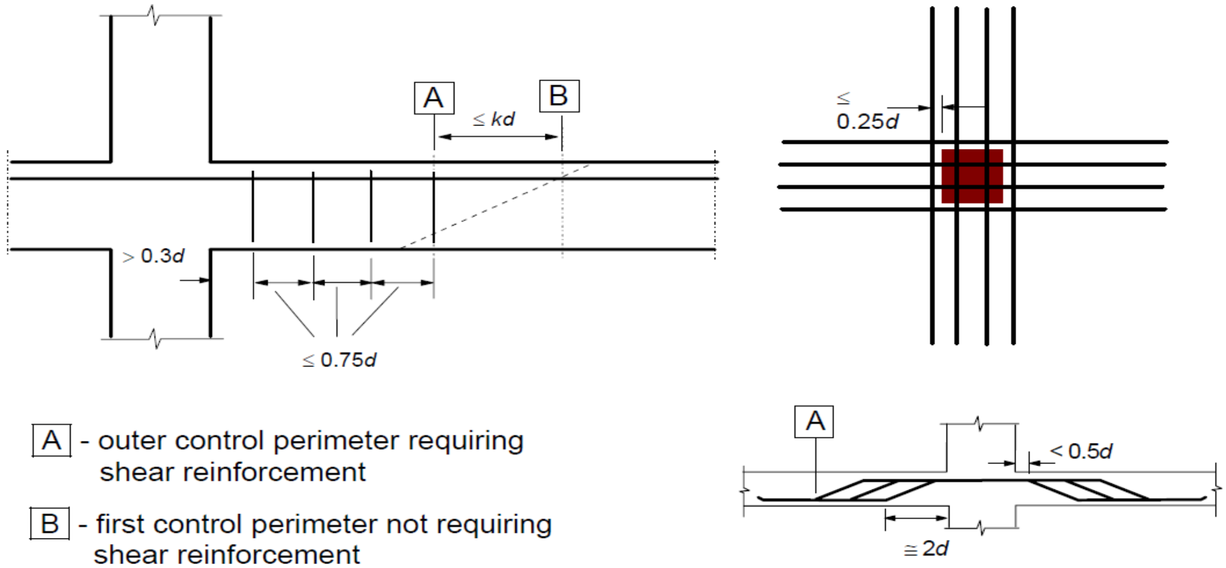
Where shear reinforcement is required the area of link leg (or equivalent) is given by

(1.5 sin

Where” = is the angle between the shear reinforcement and the main steel (i.e the vertical links) =1

is the spacing of shear links in the radial direction

Spacing of the shear links in the tangential direction



a) **Spacing of links** b) **Spacing of bent-up bars**

Fig-5.2.Punching shear reinforcement

Note: See 6.4.5 (4) for the value of k.

5.4.1.1.Load distribution and basic control perimeter:

(1) The basic control perimeter u1 may normally be taken to be at a distance 2.0d from the loaded area and should be constructed so as to minimize its length

The effective depth of the slab is assumed constant and may normally be taken as:

Wheredy and dz are the effective depths of the reinforcement in two orthogonal directions.

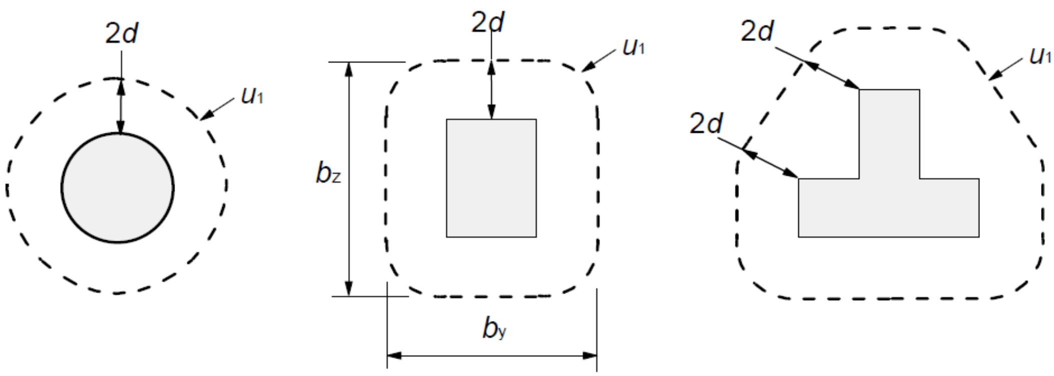


Fig-5.3: Control perimeter near an opening

The basic control perimeter is set at 2d from the loaded area. The shape of control perimeter has rounded corners.

The control perimeter at which shear reinforcement is not required should be calculated from

The outermost perimeter of shear reinforcement should be placed at a distance not greater than "kd" (k=1.5) within the outer control perimeter.

EC-2 Table 1.1

Location	Negative moments	Positive moments
Column strip	60-80%	50-70%

Middle strip	40-20%	50-30%

Table-5.2.Distribution of design moments:

Notes: The total negative and positive moments to be resisted by the column and middle strips together should always add up to 100%

The distribution of design moments given in BS8110

Column strip: hogging 75%, sagging 55%

Middle strip: hogging 25%, sagging 45%

Chapter-6

Columns

6.1. Introduction

Definition: Column or strut is a vertical compression member the effective length of which exceeds three times its least lateral dimension.

The following are applicable for h< 4b

h-larger dimension of column

b-smaller dimension of column

Columns:

Criteria	EBCS -2 New draft	EC-2	BS 8110	IS 456-2000
Main bars in compression				
	Clause 9.5.1(2) 0.10	0.10	0.004bh	0.8% of gross cross sectional area
	Clause 9.5.1(3) 0.04 - outside lap 0.08 - at laps	0.04bh	0.06bh	6% gross cross sectional area
Links:				
Minimum size	Clause 9.5.3(1) Max of(0.25 or 6mm)	0.25 or 6mm	0.25 or 6mm	Should not less than of largest diameter of longitudinal bars or 5 mm
	Clause9.5.3(4) particular special sections min(12 150mm from	min(12 150mm from restrained main bar	12 150mm from restrained main bar	Min (Least lateral dimension of the column , 16 timesthe diameter of the

	restrained main bar			smallest longitudinal bar , 300 mm)

Table 6.0 Column values

BS 8110:

Main reinforcement:

In columns in addition to minimum percentages of steel there is a requirement for a minimum number and diameter of bars.

In rectangular columns number of bars diameter of bars

In circular columns number of bars and diameter of the bars

Links:

In case of columns that when part or all of the main reinforcement is required to resist compression the links should satisfy the following clause 3.12.7.1

a) Link diameter or
b) Link spacing   12 * diameter of the smallest compression bar.

These restrictions limit the buckling length of the main compression bars and also prevent the steel from bursting out of the concrete.

Slenderness limits for columns: Clause 25.3 IS 456- 2000):

a) The unsupported length shall not exceed 60 times the least lateral dimension of the column, (.
b) If in any given plane, one end of the column is unrestrained ,   b- width at that section, and D- depth at that section.

Maximum areas of steel (Clause 3.12.6.2 BS 8110):

a) In vertically cast columns
b) In horizontally cast columns

c) Laps in vertically – horizontally cast columns

Note: The higher value in “b” reflects the better access for the placing and vibrations of concrete in horizontally cast columns.

Fire resistance requirements for columns (Clause 4.3.1, part-2, BS 8110):

The fire resistance of a column depends on its minimum dimension and the concrete cover.

Fire rating (hours)	**Minimum dimension**		**Concrete cover to reinforcement**
	Fully Exposed	**50% Exposed**	
1	200	160	25
2	300	200	35
3	400	300	35
4	450	350	35

Table 6.1 Fire resistance for columns

6.2. Column Reinforcement

6.2.1 Longitudinal reinforcement:

Longitudinal bar minimum dia=8mm

Total amount of longitudinal reinforcement >

Where- is the design yield strength of the reinforcement

Design axial compressive force.

Longitudinal reinforcement area <

Minimum number of bars in a circular column is “4”.And for columns having polygonal cross section at least one bar should be placed at each corner.

Where direction of longitudinal bars changes (at changes in column sizes) more than 1:12 the spacing of transverse reinforcement should be calculated.

Reinforcement in columns (IS-456-2000):

The cross section area of longitudinal reinforcement shall not be less than 0.8% nor more than 6% of the gross cross sectional area of the column.

The use of 6% reinforcement may involve practical difficulties in placing and compacting of concrete hence lower percentage is recommended. Where bars from the columns below have to be lapped with those in the column under consideration the percentage of steel shall usually not exceed 4%.

In any column has larger cross sectional area than that required to support the load the minimum percentage of steel shall be based upon the area of concrete required to resist the direct stress and not upon the actual area.

Spacing of longitudinal reinforcement measured along the periphery of the column shall not exceed 300 mm (Helically reinforced column)

The pitch of the helical reinforcement should satisfy the following requirement, it its strength is enhanced by 1.05

a) not more than 75 mm
b) not more than core diameter of helix bars.
c) not less than 25 mm
d) not less than 3 times the diameter of helix bars

6.2.2. Transverse reinforcement:

It may be any of the forms like links, loops or helical spiral reinforcement.

The diameter of transverse reinforcement, greater of the following

a) > 6mm
b) f the maximum dia of longitudinal bar.

The diameter of the wires of welded mesh fabric for transverse reinforcement should not be less than 5mm.

6.2.2.1. Spacing:

The spacing of transverse reinforcement along the column <

=min

b-least dimension of column

Should be reduced by a factor 0.6

a) In sections within “b” above or below a beam or slab

b) Near lapped joints where

A min of 3 bars is required in lap length.

Pitch

Fig-6.0.Column reinforcement restraining

No bar within a compression zone should be further than 150mm from a restrained bar.

Every longitudinal bar or bundle of bars in a corner should be held by transverse reinforcement.

AS3600-2009 Requirements for restraining single longitudinal bars in columns:

i) Every corner bar

ii) All bars where bars are spaced at centers > 150mm

iii) At least every alternative bar, where bar centers 150mm

For bundled bars, each bundle must be restrained.

Offset between column faces :(ACI 7.6.3.):

Where there is a change in size of a column. The slope of the inclined portion providing the offset shall not exceed one in six (1 in 6). Where column verticals are offset bent, additional ties are required and shall be placed not more than 6 in (150mm) from the point of the bend. For practical purposes 3-closely spaced ties are usually used, one of which may be part of the regularly spaced ties plus two extra ties.

Change bar arrangement between floors:-

When the bar arrangement is changed at a floor the bars may extend through terminate or require separate dowels. Reinforcing steel at least equal in area to that in the column above must be extended from the column below to lap bars above by the required lap length or butt splices must be provided vertical bars from the column below, terminated for any reason are cutoff within 3in (75mm) of the top of the finished floor.

6.3.Walls:

6.3.1. Introduction

This clause refers to reinforced concrete walls, length to thickness ratio

For walls subjected predominantly to out of plane bending the rules of slabs apply.

Clause 3.12.6.3 BS 8110:

Note: The subscript "sc" indicates the total area of main steel not necessarily steel in compression

6.3.2. Reinforcement:

6.3.2.1. Vertical reinforcement:

The area of vertical reinforcement shall lie between

utside lap locations

at laps.

Where the minimum area of reinforcement controls in design half of this area should be located at each face.

Spacing:

Distance between two adjacent vertical bars:

Minimum of the following

1) < 3 times wall thickness or
2) 400mm

6.3.2.2 Horizontal reinforcement:

It is running parallel to the faces of the wall (and to the free edges) should be provided at each surface.

It should not be less than

Spacing:

Between two horizontal bars < 400mm

6.3.2.3. Transverse reinforcement:

Where total vertical reinforcement in the two faces exceeds links required as for columns.

The larger dimension shall < 4 times thickness of well.

Where the main reinforcement is placed nearest to the wall faces transverse reinforcement should also be provided in the form of links with at least 4 per square meter of wall area.

Transverse reinforcement need not be provided where welded wire mesh and bars of diameter are used with the concrete cover larger than 2.

6.3.3. Foundations of pile caps:

6.3.3.1.Reinforcement;

Reinforcement in a pile cap should be calculated either by using strut and tie or flexural methods as appropriate.

Minimum bar diameter = should be provided = 8mm

The main tensile reinforcement to resist the action effects should be concentrated in the stress zones between the tops of piles .If the area of this reinforcement is at least equal to the minimum reinforcement evenly distributed bars along the bottom surface of the members may be omitted. Also the sides and the top surface of the member may be unreinforced if there is no risk of tension developing in these parts of the member.

The compression caused by the support reaction from the pile may be assumed to spread at 45^0 angles from the edge of the pile. This compression may be taken into account when calculating the anchorage length.

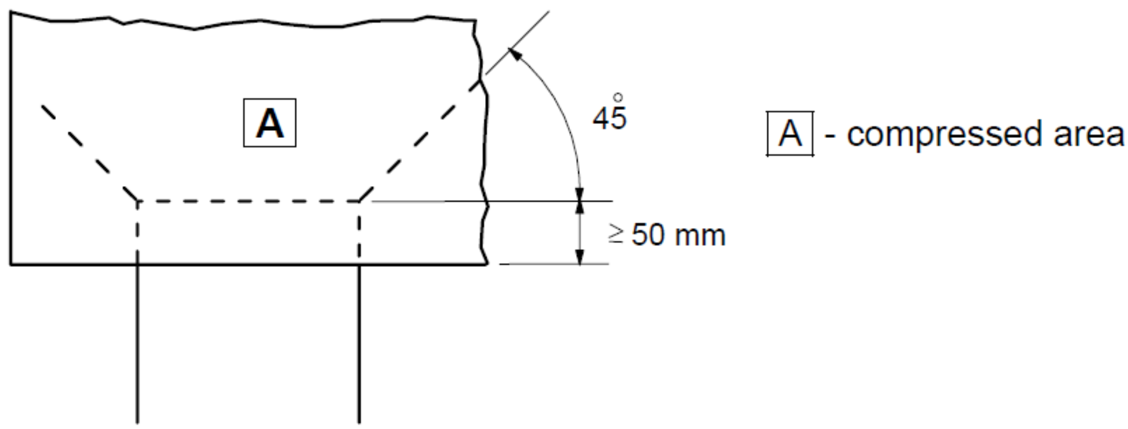


Fig6.1.Compressed area increasing the anchorage capacity

6.3.4.Columns and wall footings:

6.3.4.1.Reinforcement:

Minimum dia. of reinforcement: =8mm

The main reinforcement of circular footings may be orthogonal and concentrated in the middle of the footing for a width of 50% to 10% of diameter of footing. In this case the unreinforced parts of the element should be considered as plain concrete for design purposes.

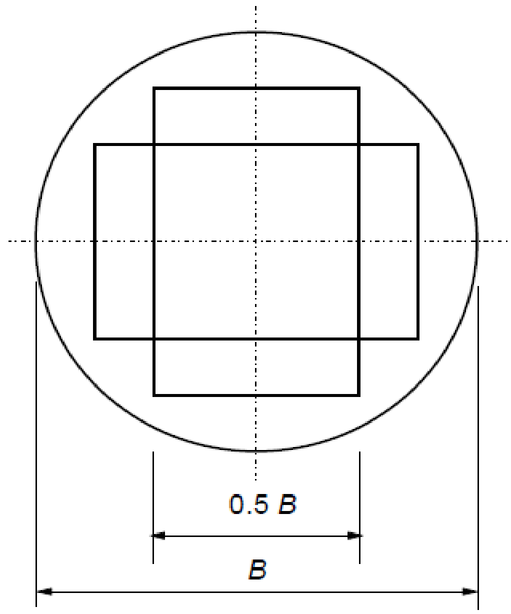


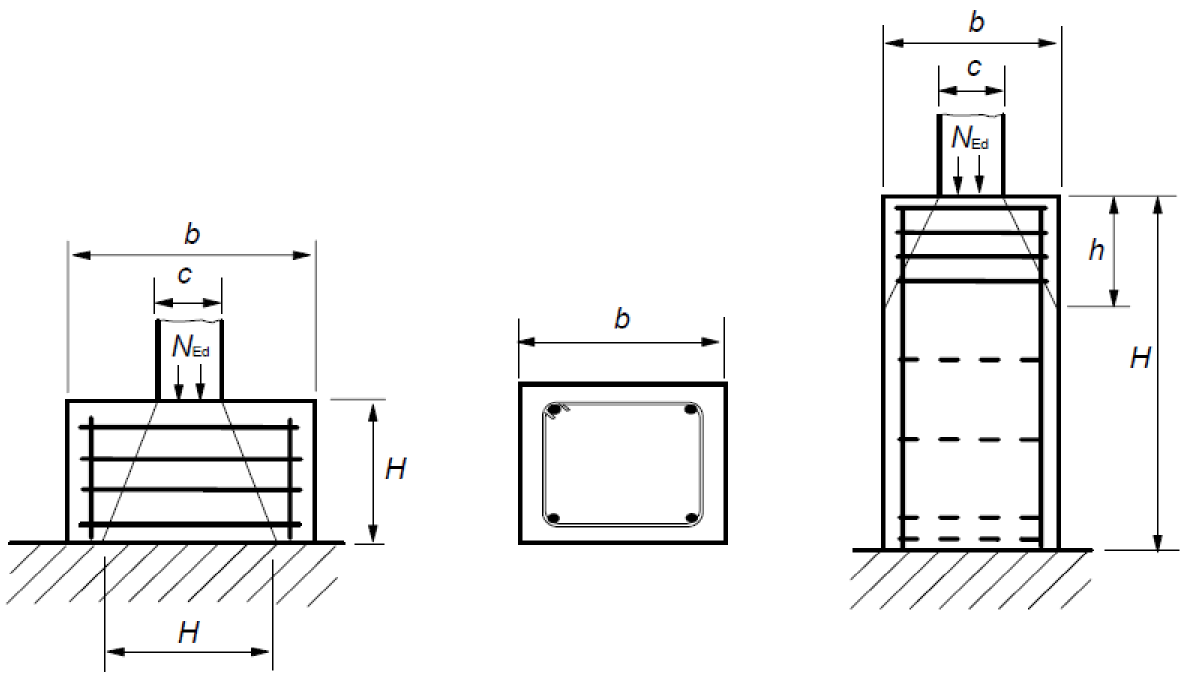
Fig-6.2.Orthogonal reinforcement in circular spread footing on soil

If the action effects cause tension at the upper surface of the footing the resulting tensile stresses should be checked and reinforced as necessary.

Tie beams may be used to eliminate the eccentricity of loading of the foundations. The beams should be designed to resist the B.M's and S.F's.
A minimum bar diameter for the reinforcement resisting B.M's should be provided.
The beams should also be designed for a minimum downward load of if the action of compaction machinery can cause effects to the tie beams.
Column footing on rock:
Adequate transverse reinforcement should be provided to resist the splitting forces in the footing when the ground pressure in the ultimate state exceeds " This reinforcement may be distributed

uniformly in the direction of the splitting force over the height “h”. A minimum bar dia should be provided. The splitting force may be calculated as follows.

Where “h” is lesser of “b” and “h”



a) Footing with h ≥ H **b) section** **c) Footing with h < H**

Fig 6.3: Splitting reinforcement in footing on rock

6.3.5. Bored piles:

For reinforced bored piles

Minimum longitudinal reinforcement related to piles cross section related to pile cross section reinforcement should be distributed along the periphery of the section

Pile cross-section Ac	Minimum area of longitudinal reinforcement: As,bpmin
$Ac \leq 0.5$ m2	$As \geq 0.005 \cdot Ac$
0.5 m2<Ac < 1.0 m2	$As \leq 25$ cm2
$Ac > 1.0$ m2	$As \geq 0.0025 \cdot Ac$

Table6.2.Recommended minimum longitudinal reinforcement area in cast in place bored Piles.

Longitudinal bar minimum dia16mm, piles should have at least 6-longitudinal bars.The clear distance between bars < 200mm measured along the periphery of the pile.

6.3.6. Cast in place piles:

In the absence of other provisions the diameter used in the design calculations of cast in place piles without permanent casing should be taken as (clause 2.3.4.2(1))

If $d_{nom} < 400$ mm $d = d_{nom} - 20$mm

If $400\text{mm} \leq d_{nom} \leq 1000$mm $d = 0.95\ d_{nom}$

If $d_{nom} > 1000$mm $d = d_{nom} - 50$mm

d_{nom}- is the nominal diameter of the pile.

Chapter-7

Serviceability limit states

7.1. Introduction

From the past century material strengths continued to increase. At the same time the overall safety factor used in design has been reducing. Result of this development makes the ultimate limit state is no longer critical. Design now often depends on serviceability limit states.

Common serviceability limit states are covered under this section: These are

1) Stress limitation
2) Crack control
3) Deflection control.

***Limitation*: ** Vibration control not covered in this standard.

***Assumptions*: ** In the calculation of stresses and deflections.

a) Cross section shall be assumed to be uncracked provided that the flexural tensile stress does not exceed

b) The value of may be taken as or provided that the calculation for minimum tension reinforcement is also based on same value.
c) For purposes of calculating crack widths and tension stiffening should be used.

7.2. Stresslimitation (Clause 7.2)

7.2.1. Concrete:

To avoid longitudinal cracks, micro cracks or high levels of creep, the compressive stress in concrete shall be limited, where they could result in unacceptable effects on the function of the structure.

Longitudinal cracks:

An area exposed to environments of exposure classes .Longitudinal cracks may lead to a reduction of durability these occurs under the characteristic combination of loads exceeds a critical value. If other measures are absented such as increase in the cover to reinforcement in the compressive zone or confinement by transverse reinforcement, it may appropriate to limit the compressive stress to a value in areas exposed to above said.
is given in national annex=0.6

Creep:

Liner creep and nonlinear creep.
If the stress in concrete under quasi permanent loads
- Linear creep may be assumed.
Nonlinear creep considered
National Annex=0.45

7.2.2. *Steel*:

Tensile stresses in reinforcement shall be limited
1. To avoid inelastic strain
2. Unacceptable cracking or deformation.

Unacceptable cracking or deformation:

For the appearance unacceptable cracking or deformation may be assumed to be avoided if

a) Under the characteristic combination of loads the tensile stress in reinforcement does not exceed
b) Where the stress is caused by an imposed deformation the tensile stress should not exceed

- National Annex- 0.8 and 1.0 respectively.

7.3. Crack control (Clause 7.2):

7.3.1. General considerations (Clause 7.3.1)

Cracking is normal in reinforced concrete structures subject to bending, shear or torsion or tension resulting from either direct loading or restraint or imposed deformations. Cracking shall be limited to an extent that will not impair the proper functioning or durability of the structure or cause its appearance to be unacceptable. Cracks may be permitted to form without any attempt to control their width provided they do not impair the functioning of the structure.

7.3.2. Limitation:

Other cause for cracks such as plastic shrinkage or expansive chemical reactions within the hardened concrete –Their avoidance and control lie outside the scope of this section.

A limiting value for the calculated crack width " taking into account the proposed function and nature of the structure and the costs of limiting cracking should be established.
given in national annex.

EBCS-2, 2013(Draft)-Table 7.1N. Recommended values of (mm)

Exposure Class	Reinforced members and prestressed members with unbounded tendons	Prestressed members with bonded tendons
	Quasi-permanent load combination	**Frequent load combination**

X0,XC1	0.4[1]	0.2
XC2,XC3,XC4	0.3	0.2[2]
XD1,XD2,XD3, XS1,XS2,XS3		Decompression
Note 1: For X0, XC1 exposure classes, crack width has no influence on durability and this limit is set to give generally acceptable appearance. In the absence of appearance conditions this limit may be relaxed. **Note 2:** For these exposure classes, in addition, decompression should be checked under the quasi-permanent combination of loads.		

Table-7.0. Recommended values of (mm)

Verification in SLS (Service limit state) may also be carried out using strut and tie models 'e.g' verification of steel stresses and crack width control, if approximate compatibility for strut and tie models is ensured (In particular the position and direction of important struts should be oriented according to linear elastic theory).When using strut and tie models with the struts oriented according to the compressive stress trajectories

7.3.3. Flexural cracking: AS3600-2009

Crack spacing "s" varies between 0.5d and 1.5d and depends on

i) Steel area and distribution
ii) Cover

The average crack spacing decreases with time due to shrinkage.

Crack width "w" depends on

i) Steel stress
ii) Bar diameter and bar spacing
iii) Cover
iv) Adjacent crack spacing's

It increases with time due to shrinkage. Maximum crack widths increase with time by a factor of between 2 and 4.

Simplified approach for flexural crack control in As3600-2009(Clause 8.6.1 and 9.4.1):
For reinforced concrete beams and slabs cracking is deemed to be controlled (Crack widths will be less than 0.3 mm) if each of the following is satisfied.

i) The distance from the side or soffit of the member to the Centre of the nearest longitudinal bar shall not exceed 100mm.
ii) The Centre to Centre spacing of bars near a tension face of beam or slab shall not exceed 300mm. for a beam and the lesser of the two times the slab thickness and 300mm for slab.
iii) The Quantity of tensile reinforcement in abeam or slab provides an ultimate tensile strength at least 20% higher than the cracking moment.
iv) The stress in the tensile steel is less than a limiting value.

7.4. Deflection control (Clause 7.4):-

The deformation of a member or structure shall not be such that it adversely affects it's proper functioning or appearance .Appropriate limiting values of deflection taking into account the nature of the structure of the finishes, partitions and fixings and upon the function of the structure should be established.

Derived from ES ISO 4356:-

1) **From appearance and general utility:-**

   The calculated sag of a beam slab or cantilever subjected to quasi permanent loads deflection, otherwise appearance and general utility of the structure is impaired. Pre camber may be used to compensate for some or all of the deflection but any upward deflection incorporated in the formwork should not generally exceed, The sag is assessed relative to the supports.

2) **From deflection that damage adjacent parts:**

For the deflection after construction is normally an appropriate limit for quasi permanent loads.

The limiting Span/depth ratio may be estimated using following expressions and multiplying this by correction factors to allow for the type of reinforcement used and other variables. No allowance has been made for any pre camber in the derivation of these expressions.

-----(a)

--------(b)

Where:

l/d is the limit span/depth

K is the factor to take into account the different structural systems

ρ_0 is the reference reinforcement ratio =

ρ is the required tension reinforcement ratio at mid-span to resist the moment due to the design loads (at support for cantilevers)

ρ' is the required compression reinforcement ratio at mid-span to resist the moment due to design loads (at support for cantilevers)

f_{ck} is in MPa units

Expressions (a) and (b) have been derived on the assumption that the steel stress under the appropriate design load at SLS(Service limit state) at a cracked section at the mid span of a beam or slab or at the support of a cantilever is 310 Mpa (corresponding roughly to)

Where other stress levels are used the values obtained using expressions (a) and (b)should be multiplied by

Where is the tensile steel stress at mid span (at support for cantilevers) under design load at SLS.

is the area of steel provided at this section.

is the area of steel required this section for for ultimate limit state.

For flanged sections where the ratio of the flange breadth to rib breadth exceeds “3” the values of given by expression (a) and (b) should be multiplied by 0.8

For beams and slabs other than flat slabs with spans exceeding 7m which support partitions liable to be damaged by excessive deflections the values of given by expression (a) and (b) should be multiplied by (where is in meters.)

For flat slabs span greater than 8.5m supports partitions liable to be damaged by excessive deflections the values of given by expression (a) and (b) should be multiplied by (where in meters)

For the values of “k” – national Annex.

Basic ratios of span/ effective depth for reinforced concrete members without axial compression.

Structural System	K	Concrete highly stressed $\rho = 1.5\%$	Concrete lightly stressed $\rho = 0.5\%$
Simply supported beam, one – or two-way spanning simply supported slab	11.0	14	20
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	11.3	18	26

Interior span of beam or one-way or two-way spanning slab	11.5	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	11.2	17	24
Cantilever	0.4	6	8

Note 1: The values given have been chosen to be generally conservative and calculation may frequently show that thinner members are possible.
Note 2: For 2-way spanning slabs, the check should be carried out on the basis of the shorter span. For flat slabs the longer span should be taken.
Note 3: The limits given for flat slabs correspond to a less severe limitation than a mid-span deflection of span/250 relative to the columns. Experience has shown this to be satisfactory

Table-7.1.Values for "K" and "ρ"

Members which are not expected to be loaded above the level which would cause the tensile strength of the concrete to be exceeded anywhere within the member should be considered to be uncracked. Members which are expected to crack, but may not be fully cracked, will behave in a manner intermediate between the uncracked and fully cracked conditions and, for members subjected mainly to flexure, an adequate prediction of behavior is given by Expression below:

$\alpha = \zeta\alpha_{II} + (1 - \zeta)\alpha_{I}$

Where

α is the deformation parameter considered which may be, for example, a strain, a curvature, or a rotation. (As a simplification, α may also be taken as a deflection -

α_{I}, α_{II} are the values of the parameter calculated for the uncracked and fully cracked conditions, respectively

ζ is a distribution coefficient (allowing for tensioning stiffening at a section) given by Expression

ζ = 0, for uncracked sections

β- is a coefficient taking account of the influence of the duration of the loading or of repeated loading on the average strain

= 1.0 for a single short-term loading

= 0.5 for sustained loads or many cycles of repeated loading

σ_s is the stress in the tension reinforcement calculated on the basis of a cracked section

σ_{sr} is the stress in the tension reinforcement calculated on the basis of a cracked section under the loading conditions causing first cracking

Note: σ_{sr}/σ_s may be replaced by M_{cr}/M for flexure or N_{cr}/N for pure tension, where M_{cr} is the cracking moment and N_{cr} is the cracking force.

For loads with a duration causing creep, the total deformation including creep may be calculated by using an effective modulus of elasticity for concrete according to Expression:

Where:

$\varphi(\infty, t_0)$ is the creep coefficient relevant for the load and time interval (see 3.1.4)

Shrinkage curvatures may be assessed using Expression below

Where:

$1/r_{cs}$ is the curvature due to shrinkage

ε_{cs} is the free shrinkage strain (see 3.1.4)

S is the first moment of area of the reinforcement about the centroid of the section

I is the second moment of area of the section

α_e is the effective modular ratio

$\alpha_e = E_s/E_{c,eff}$

S and *I* should be calculated for the uncracked condition and the fully cracked condition, the final curvature being assessed by use of Expression

7.5. Environmental conditions (Given in Annex-A)

(1) P Exposure conditions are chemical and physical conditions to which the structure is exposed in addition to the mechanical actions.

(2) Environmental conditions are classified according to Table 4.1, based on ES-EN 206-1.

(3) In addition to the conditions in Particular forms of aggressive or indirect action should be considered including: Chemical attack, arising from e.g.

i) the use of the building or the structure (storage of liquids, etc)

ii) solutions of acids or sulfate salts (ES-206-1, ISO 9690)

iii) chlorides contained in the concrete (ES-EN206-1)

iv) alkali-aggregate reactions (ES-EN 206-1, National Standards)

Physical attack, arising from e.g.

i) temperature change

ii) abrasion

iii) Water penetration (ES-EN206-1).

Discussion

i) Regarding development length:: The EBCS code book s silent on the development length of the bars in contact(i.e bundled). Therefore in the bundled bars the development length of each bar shall be that for the individual bar , increased by 10% fro two barsin contact, 20% for three bars in contact and 33% for four bars in contact. The anchorages of the bars of a bundle can only be straight anchorages.

ii) Splicing in case of bundled bars, lapped splices of bundled bars shall be made by splicing one bar at a time, such individual splices within a bundle shall be staggered .For bundles of 2, 3, or 4 bar the staggering distance should be 1.2, 1.3 and 1.4 times the anchorage length of the individual bars respectively.

iii) Where more than half of the bars are spliced at section or where splices are made at points of maximum stress, special precautions shall be taken such as increasing the length of the lap and or using spirals or closely spaced stirrups around the length of the splice.

iv) Regarding the side face reinforcement: Where the depth of the beam exceeds 750mm in case of beams without torsion and 450mm with torsion side face reinforcement shall be provided. Side face reinforcement shall be provided along the two faces. Total area of such reinforcement shall not be less than 0.1% of web area distributed equally on two faces spacing not exceeding 300mm or b.

v) About reinforcement provision in two way slabs i.e., main and distribution. In two way slabs reinforcement parallel to the short span of the slab shall be placed in the bottom layer at mid span and in the top layer at support.

vi) EBCS has provided how to provide the reinforcement in beam column connection at the end support but it was not provided about the beam column at intermediate and beam-beam intersection for the provision of main reinforcement. Therefore at beam column intersections ensure that the main beam bars avoid the main column bars. And at beam-beam intersections main reinforcement may be so arranged that layers in mutually perpendicular beams are at different levels. Thus to accommodate bottom bars it is good practice to make secondary beams shallower than main beams at least by 50mm.

vii) In columns for using helical reinforcement a reinforced column shall have at least six bars of longitudinal reinforcement.

viii) Column connection with the footing: Column bars of diameter larger than 36mm in compression can be spliced with dowels at the footing with bars of smaller sizes and of necessary area. A dowel shall extend into a column a distance equal to the development length of the column bar and into footing distance equal to development length of the dowel. Where dowels are provided their dia. shall not exceed the diameter of the column bars by more than 3mm.

ix) Flexural reinforcement preferably shall not be terminated in tension zone. If such bars larger than 36mm lap splices shall not be used and also shall not be bundled.

x) Curtailment: Bars in bundle shall terminate at different points spaced apart by not less than 40 times the bar diameter except for bundles stopping at a support.

xi) Force not applied to top of beam: Where a load transfer is through the bottom or side of beam (for example where one beam frames into another) ensure that there is sufficient suspension or hang up reinforcement at the unction in the main beam in the form of stirrups to transfer the force to the top of the beam. If the load is large bent up bars may also be used in addition to stirrups.

xii) Development length of the bars in tension or compression: The calculated tension or compression at any bar at any section shall be developed on each side of the section by an appropriate development length or end anchorage or by a combination thereof.

xiii) Fritz Leonhardt (1974)(Famous Professor) presents the following suggestions of appropriate and inappropriate solutions for lap splices in order to avoid high concentration of tensile stresses in the concrete around the bar that might cause splitting cracks in concrete.

Inappropriate Lap Splice

The stress - here corresponds to the transversal component related to the bond stress.

Appropriate Lap Splice

Conclusion:

Though the EBCS also includes certain guidelines for the Detailing, the code majorly deals with the design principles of the various members. The scope of this these is so wide , even then I tried to incorporate which are very much required for detailing from EBCS new draft along with the developed countries and developing countries.

Future work:

For future scope of the study

i) Can be further elaborated with the other detailing regarding stair cases , beam column and beam-beam joints

ii) Can be applied for pre stress concrete

iii) Similarly for the earthquake code of EBCS-8 also can be formulated as detailing code.

Annex-A

Exposure classes related to environmental conditions in accordance with ES-EN 206-1

Class Designation	Description of the Environment	Informative examples where exposure classes may occur
1. No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2. Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3. Corrosion induced by chlorides		

XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides ,pavements car park slabs
4. Corrosion induced by chlorides from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
5. Freeze/Thaw Attack		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surface exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agents	Horizontal concrete surfaces exposed to rain and freezing
XF4		Road and bridge decks exposed to de-icing

	High water saturation with de-icing agents or sea water	agents Concrete surfaces exposed to direct spray containing de-icing agents and freezing Splash zone of marine structures exposed to freezing
6. Chemical attack		
XA1	Slightly aggressive chemical environment according to ES-EN 206-1,Table 2	Natural soils and ground water
XA2	Moderately aggressive chemical environment according to ES-EN 206-1, Table 2	Natural soils and ground water
XA3	High aggressive chemical environment according to ES-EN 206-1, Table 2	Natural soils and ground water

References:

1) ACI 318M-2005, Building code requirements for structural concrete (Metric)

2) Australian code AS3600-2009 of.

3) Bureau of Indian Standard, Hand book on concrete reinforcement and detailing, SP-34-1987

4) Concrete society, Standard method of Detailing structural concrete, 3rd edition.

5) Department of Transport and main roads drafting and Design Presentation standards manual, State of Queens land(Department of Transport and main roads) 2011

6) Ethiopian Building Code Standard Draft code-2013.

7) EURO CODE-2 But it is supplemented by number of other standards the following in Particular but they are not referred, but quoted for educate the reader (A. EN10080- steel for the reinforcement of concrete (1). B. EN ISO 17760 – Permitted welding

process for reinforcement (2), C.EN ISO 3766:2003- construction drawings simplified representation of concrete reinforcement (3), D.EUROCODE-8- Design of structures for EQ resistance (4)

8) J.E.Breen, J.O.Jirsa,R.Bergmeiter, andM.E.KergerDetailing for structural concrete (Study conducted in cooperation with the U.S Department of Transportation, Federal Highway Administration)

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11) Mark Fintel,CBS Publishersand distributorsHand book of concrete engineering, 2nd edition.

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